

**Investigation of a Tall Building Structure:
The Spiral Building**

by

Leora Sasson

Submitted to the Department of Civil and Environmental
Engineering in Partial Fulfillment of the Requirements
for the Degree of

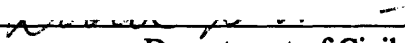
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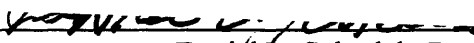
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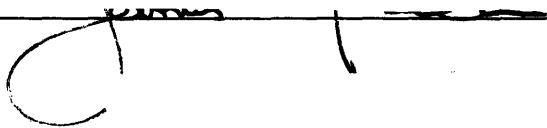
February 1995

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ABSTRACT

This thesis investigates the design of the *Spiral Building*. The building may be classified as a tall building. The process and procedures of performing a preliminary design of a tall building are discussed first, including a discussion of building loads, force flow, building structural components, and building structural systems and their behavior under loading. Construction effects on design are then briefly discussed. Specific structural systems are discussed in relation to their relevance to the *Spiral Building*. This is followed by a brief discussion of precedent buildings in relation to the *Spiral Building*.

The core of this thesis consists of the preliminary analysis of three structural systems for the purpose of accommodating the *Spiral Building* form. Upon completion of the analyses, the three alternatives are individually evaluated and then compared against each other. Ultimately, one final optimum solution is selected based on the evaluations and comparisons.

The final part of this thesis consists of a brief discussion of the functional requirements of a tall building, including occupancy and exit requirements. A brief study of the functional requirements of the *Spiral Building* is performed as well a study of some possible floor layouts for various areas of the building.

Thesis Advisor: Daniel L. Schodek, Professor of Architecture, Harvard Graduate School of Design

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PART 1

The Spiral Building: Introduction

CHAPTER 1

The Spiral Building

In this paper, an unusual tall building form will be investigated. In general, this form will consist of a typical cruciform plan rotated 10° at each level to form a spiral (figure 1.1). It may be visualized as four spiral staircases climbing on one central column. The purpose for this investigation is not out of a *need* for a spiral building, but rather out of a search for

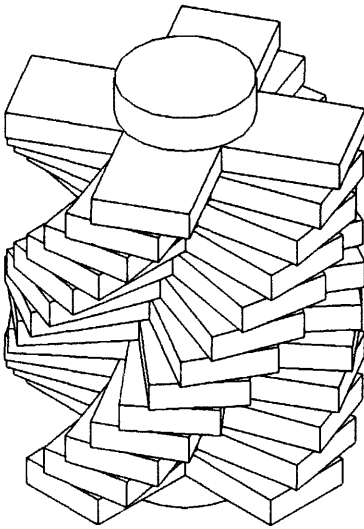


Figure 1.1: Building model

innovative architectural forms and technologies. The structural requirements of such a building will require special considerations and a non-traditional type of structural system. Though the required structural system may not be innovative, it will involve a challenging design process. The building form was developed in an attempt to get away from the high-rise glass box forms that are so common today. It shall be noted here that this building concept was developed by the author of this paper independently and any similarity of this building form to any existing building or building idea is purely coincidental. Following, are some general descriptions of the building and its use.

1.1 Building Occupancy

The occupancy of the spiral building will be residential and will include recreation facilities. Each floor level will consist of four units, each unit being isolated from adjacent units on all sides.

1.2 Physical Description

The spiral building will consist of a ground floor which will house the lobby, 13 floor

levels which will comprise the spiral, and a mechanical room at the top (figure 1.2). The first floor will house the storage room, recreation room, lounge, and exercise room (figure 1.3).

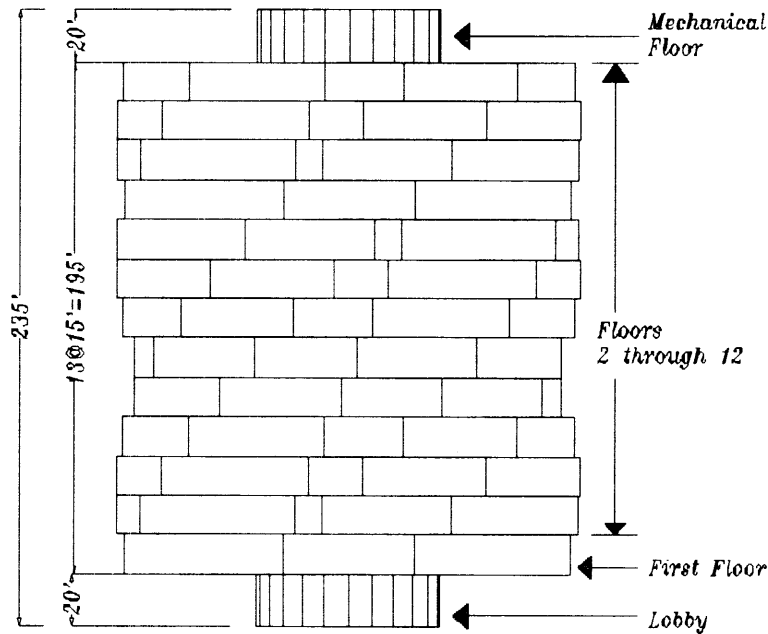


Figure 1.2: Building elevation

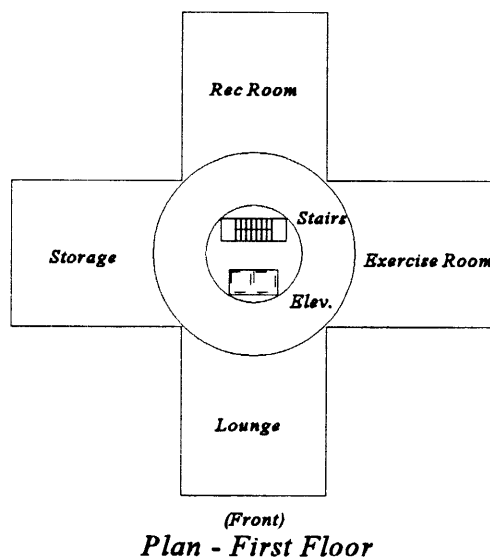


Figure 1.3: First floor plan layout

The lobby level will be enclosed in the boundaries of the building core as will the mechanical room. The first level of the spiral will begin approximately 20 feet above the ground and will overhang from the core over an outdoor area. Each 10° step of the spiral will consist of a cruciform shape in plan, or four individual blocks attached to the core. Each of these blocks will house one individual living unit and will enclose the minimum dimensions of 50 feet by 50 feet, or 2500+ square feet per unit. The spiralling of the building will create large

open outdoor terraces of approximately 500 square feet and a sense of privacy for each unit.

The central core of the building will be circular in plan with an outside diameter of 70 feet. At each floor level, the core will allow access into each of the four units (figure 1.4). Within the main structural core will be a secondary service core which will house all the vertical building systems such as elevators, exit stairs, and mechanical and plumbing systems. The top level of the core will protrude 20 feet above the roof of the thirteenth floor and will house the mechanical rooms for the building services. Each unit will have its own mechanical systems for HVAC, etc.

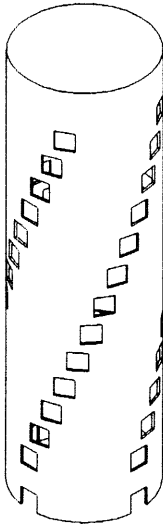


Figure 1.4: Core model

Commonly, a building is designed for a specific site. In this case, however, there was no specific site in mind when the idea was developed. Given the large open terraces, an ideal location would be in an area with warm weather and plenty of sunshine. However, given the building form, the structure may not perform well in a high seismic risk area. Thus, the general location chosen for the spiral building is along the coast of Florida, and the loadings will be chosen accordingly.

CHAPTER 2

Goals

A structural system is expected to:

- Carry dynamic and static vertical loads.
- Carry horizontal loads due to wind and earthquake.
- Resist and help damp vibrations and fatigue effects.
- Conform with the requirements of the architect and the user.
- Facilitate simple and fast erection of building.
- Be economical.¹

In the process of selecting the most suitable structural system for a tall building, these factors must be considered and optimized. This process is usually complicated and there does not exist one clear-cut method for selecting an appropriate structural system.

A structural investigation must progress with the following considerations:

- Determination of the structure's physical being with respect to the materials, form, scale, detail, location, support conditions, and internal character.
- Determination of the load demands placed on the structure.
- Determination of the structure's response in terms of deformations and development of stresses.
- Determination of the limits of the structure's capabilities.
- Evaluation of the structure's effectiveness.²

Most often, there does not exist a single ideal solution to a given structural problem, however, there does exist an optimum solution. Several structural systems should be investigated and evaluated, and then compared against each other in terms of their structural effectiveness as

1

Tall Building Systems and Concepts, Monograph on Planning and Design of Tall Buildings, Vol. SC, 1980, pp. 3.

2

Ambrose, J., Building Structures, Second edition, John Wiley and Sons, Inc., 1993, pp. 47.

well as their overall costs, ease of erection, simplicity of analysis, etc.

This paper will discuss the general processes involved in preparing for and implementing a design of a tall building. Then an investigation will be made into the design of the *Spiral Building* consisting of a preliminary design of three structural alternatives. The analyses will include loading conditions, structural responses and design, and some functional considerations. Upon completion of the analyses, the three solutions will be compared against each other so that the optimum one may be selected. The references that will be used in the structural design include the Uniform Building Code (UBC), the American Institute of Steel Construction Manual (AISC), and the American Concrete Institute Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-89). References for the functional requirements will include the UBC and Architectural Graphic Standards.

PART 2

Criteria Involved in the Design of a Tall Building

CHAPTER 3

Tall Building Theory: Past and Present

The design of a tall building is based on a complex interactive process. The many form determinants include the effect of the physical surroundings, the evolution in technologies and styles of the high-rise building form, and the proper functioning of the building systems. A design requires a comprehensive investigation by a team of individuals into these various facets that surround an idea. This team is comprised of owners, architects, engineers, contractors, and city officials, just to name a few. Cooperation among these individuals is necessary for the successful arrival at a completed design.

In early times, tall buildings included terraced temple mounts, pyramids, fortresses, and temples. These structures were symbols of power. The Romans built 10-story tenement buildings, mainly of wood. Later building height was limited to 70 feet by Emperor Augustus to reduce the risk of fire, an early example of enforcing a building code. Through the Middle Ages, common multistory building materials included stone and brick used in wall construction and, in combination with wood, used in such structural systems as arches, vaults, and post-and-beam construction. In the 19th century, lighter flooring systems became possible with the use of hollow bricks, clay tiles, and eventually, with the introduction of reinforced concrete slabs.

The traditional tall masonry buildings were gravity structures. Their increased weight with height set a limit on height so that the proportions of structure area to floor area at the base would not be unreasonable. For example, the 16-story Monadnock Building (1891) in Chicago had structural masonry walls occupying nearly one-fifth of the floor area at the base. In the early 19th century, the metal skeleton began to replace the heavy masonry construction of multistory buildings. This was an important component in the evolution of the skyscraper because it allowed for much more flexibility in plan and thus, height, while retaining structural integrity. Further evolution of the skyscraper came with the development of the passenger elevator, which allowed for buildings to attain heights of more than the previously limited five stories.

Architectural considerations are important in the design of any building. Architecture is an art as well as a science; it translates abstract ideas into physical form. Architecture depends on the unpredictable response of the human being out of his needs and feelings, for which there is no measure. Furthermore, architecture depends on the current situation, but

at the same time derives its expression from the past. Thus, it is necessary to attain a balance between tradition and innovation. In earlier times, styles lasted for long periods of time, testifying to the fact that architectural concepts were unified. Today there are many architectural theories, as expressed in the diversity of styles, owing primarily to the development of new building materials and construction methods. The richness and limitless potential of individual buildings is suggested in the study of unconventional architecture. "However, all modern theories are still based on the principles of Vitruvius that a building must have: functional serviceability, strength, safety, durability, economy, and an appealing appearance." ¹

¹

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp. 16-18.

CHAPTER 4

Structural Concepts

A building structure makes enclosure possible by preventing the building from collapsing by resisting gravity and lateral loads. Structures range from massive gravity blocks to slender pure structures. The solid mass of a gravity structure is inherently stable in resisting lateral force action whereas a tall slender building provides the bare minimum of structure. The slender tower acts a cantilever fixed at the ground and must resist lateral forces by using all of its energy in bending. As building height increases, lateral load actions rather than gravity load action becomes the more dominant consideration in design. In other words, the stiffness of the structure becomes more dominant than the strength. A building property useful in determining what type of load action governs is the slenderness ratio, which is the ratio of the building height to the horizontal dimension of the support system. As the slenderness ratio increases to approximately 8:1, stiffness becomes the more dominant design issue and as the slenderness ratio increases more, the flexibility of the structure, or deflection criteria, becomes critical.

A gravity tower is one which is slender enough to buckle under its own weight before the compressive strength of the material is reached. It may be treated as a free-standing cantilever column. Such a tower with solid uniform cross section will buckle under its weight when the following compressive load is reached^{1,2}:

$$P_{cr} = \frac{7.84EI_0}{H^2}. \quad 4.1$$

When the bulk of the axial load is present at the top of the structure, its buckling capacity is lowered. Derived from Euler's formula of elastic buckling, the relation for the buckling capacity of a slender tower with uniform circular cross section is:

1

Popov, E. P., Mechanics of Materials, Second edition, Prentice Hall, Inc., 1978, pp.467-469.

2

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp. 81.

$$P_{cr} = \frac{\pi^2 ED^2}{32H^2}. \quad 4.2$$

However, overall buckling is rarely a problem, especially for ordinary buildings of normal slenderness ratio. Under certain conditions, lateral load action can be ignored in design. This condition may be obtained from the familiar interaction equation for combined axial load and bending:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1. \quad 4.3$$

Most codes (UBC, sec. 2317) allow for an increase of one-third in the allowable stresses under the action of wind or earthquake loading. Generally assuming that the allowable stresses are increased by one-third and letting $F_a = F_b$,

$$f_a + f_b \leq 1.33F_a. \quad 4.4$$

For the condition that f_a is equal to F_a , f_b must be less than or equal to $1/3f_a$. This implies when the bending stress, f_b , due to lateral loads is less than one-third the axial stress, f_a , due to gravity loads, then the effect of lateral loads may be neglected and the structure may be treated as a gravity tower.¹

A building loaded concentrically only with gravity loading will exhibit a uniform contact pressure at the base. However, in the presence of lateral forces, the stresses at the base will no longer be constant and uplift, or tension, forces may result. If the upward reaction of the combined action of the vertical and lateral loads falls within the middle one-third of the base dimension, the tension due to the lateral force action will be suppressed. If this resultant falls outside the middle one-third, then tension forces will develop at the base.¹ When the resultant falls outside the dimensions of the base altogether, the building will not

¹

Lin, T. Y., Stotesbury, S. D., Structural Concepts and Systems for Architects and Engineers, Second edition, Van Nostrand Reinhold, 1988.

be stable unless it is properly anchored to the ground. In other words, in order to have stability of the structure, the overturning moment caused by lateral force action must be resisted by the weight of the structure, the capacity of the material, and the anchorage systems establishment of continuity with the structure below any given level. According to most codes (UBC, sec. 2317), conservatively assuming that only the weight of the structure counteracts overturning moment, the resisting moment, M_r , must be at least 50% larger than the overturning moment, M_o , caused by lateral loading. In other words, the factor of safety is required to be:

$$S.F. = \frac{M_r}{M_o} \geq 1.5. \quad 4.5$$

This safety factor will yield a location of the resultant force at a minimum distance of one-sixth the base dimension from the edge of the base.

4.1 The Building Structure

A building structure may be visualized as horizontal planes (floor framing) supported on vertical planes (wall framing). The horizontal planes tie the vertical planes together to achieve a box effect and a certain degree of compactness. A slender tower must be a compact, closed structure where the entire system acts as one unit, whereas the massive building block needs only some stiff elements that give lateral support to the rest of the building. The latter represents an open system where separate stabilizing vertical planar structural systems are located at various places. A building structure consists of load-bearing structural elements and non-load-bearing elements. The load-bearing elements are the gravity structure and the lateral force resisting structure and are considered the primary structural systems of a building. The gravity structure supports only vertical loads, whereas the lateral force resisting structure must provide lateral stability in addition to supporting vertical loads. A secondary structure

resists only lateral loads. Failure of a secondary member is not as critical as failure of a main member, where immediate collapse of a portion of the building may occur. Non-load-bearing elements include building membranes that cover the structure and divide the space.

The selection of a structural system depends on several factors, including the overall building geometry, the vertical profile, and the slenderness ratio, and is a function of strength, stiffness, and ductility demands. However, it should not be interpreted that pure structural systems do not allow any flexibility in the architecture; there are endless possible combinations of structural systems. The strength and stiffness of a building is very much related to the type and arrangement of the structural elements; the continuity of the elements, together with the degree of symmetry, indicate the degree of compactness of the structure.

Although buildings are three-dimensional, their support structures may often be treated as an assembly of two-dimensional vertical planar elements in each major direction of the building. Two-dimensional structures include bearing-wall structures, framed structures, core structures that do not necessarily integrate the entire building shape, and combinations of the these. Some three-dimensional structures are staggered systems, tube structures, and megastructures.

4.2 Building Loads

The primary loads on a structure are due to the vertical action of gravity and the horizontal action of wind and earthquake. These static or dynamic, external or internal loads may represent distributed or concentrated forces and may act concentrically or eccentrically. The assumptions are as follows:

- The weight is considered uniform and a function of the floor area or building volume.
- Wind pressure is normal to the exposed surface and is considered uniformly distributed over the building height. Though it is a dynamic load, it is often treated

as a static lateral load.

- For a typical building form, the seismic force is considered to act on the building height as a static triangular load.

Though wind and earthquake forces are dynamic, the approach that they act as static loads is reasonable as long as the building is not of unusual shape or mass distribution.

4.2.1 Dead and Live Loads

Dead and live loads are gravity loads that act on any structure. Dead loads are static and remain constant and they include the weight of the structure, ceilings, flooring, partitions, mechanical systems, etc.; in other words, the weight of those items that remain constant throughout the life of the building. Those gravity loads that are not part of the dead loads are live loads. Live loads are dynamic and do not remain constant, however, because live loads are applied slowly, they are treated as static loads. Since the 1960's, buildings have become much lighter so that the effect of live loads relative to dead loads has become much more significant.

Concrete and masonry building generally weigh more than steel buildings; the overall average gross dead weight for ordinary steel buildings is approximately 50 to 80 psf, whereas reinforced concrete buildings may weigh twice as much. The use of high-strength material results in less weight, which may be advantageous when strength rather than stiffness controls the design. The weight of the structure constitutes only a small portion of the total building dead load, 20 to 50% for frame buildings and is dependent on the height. For example, a typical 10-story steel frame building may weigh as little as 6 psf whereas a 100-story steel frame building will weigh 30 psf. This effect is known from nature where animal skeletons become much bulkier with increased size since the weight increases with the cube of the dimensions, while the area increases with the square. The bones of a mouse occupy

approximately 8% of the total mass while the bones of a human occupy about 18%.¹

Live loads are not permanent on a structure. Floor live loads are caused by contents or object called occupancy loads while roof live loads are caused by snow, rain, and ice. Occupancy loads include the weight of people, furniture, etc, the values of which are specified in building codes as distributed loads. When live loads act on small areas, they may have to be considered as concentrated loads. It is apparent from the codes that public areas such as corridors must carry more live load than living or working areas and that office buildings weigh more than apartment buildings in relation to the live loads. Live load values are conservative because they are unpredictable in nature.

It is improbable that a multi-story building will experience full live load on every floor simultaneously; generally, the larger the area or number of floors, the smaller is the potential load intensity. Building codes take this into account by allowing for the use of reductions in live load on structural members. For members supporting tributary areas greater than 150 square feet, the allowable reduction in percent is:

$$R = 23(1 + D/L), \quad 4.6$$

where D and L represent the dead and live loads, respectively, and the maximum value on R is 40% for members supporting loads from only one level and 60% for other members. However, there are some restrictions on when reduction is allowed. Live load reduction is not allowed:

- if the live load is greater than 100 psf,
- if the tributary area is less than 150 square feet,
- for one-way slabs, and
- for garages, roofs, or areas of public assembly.²

1

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp. 40.

2

"Uniform Building Code," International Conference of Building Officials, 1991,

Reduction of live loads is important because it results in significant savings in material.

Roof live loads are usually smaller than floor live loads. Snow loads are identified in the codes by snow load maps and range from 5 psf to 80 psf and up to 300 psf at higher elevations. For preliminary design purposes, the following roof live loads may be assumed:

South of latitude 37°	20 psf
Between latitude 37° and 45°	30 psf
North of latitude 45°	40 psf

except at high altitudes, where 10 psf should be added to the above.¹ Snow loads are considered to act on the horizontal projected plane of inclined roofs. Water loads may become important for flat roofs where improper drainage or insufficient slope may result in ponding.

At the initial design stage, the dead weight of structural members must be estimated. The best procedure for doing this is to begin at the roof and follow the force flow through the structure. At the different design stages, the estimated loads and member sizes should be checked and adjusted so that errors do not accumulate. Placement of live load forces should be as such to create the maximum shear, bending, and axial forces on structural members and/or systems. For example, in column design, those loads causing the maximum axial forces and those causing the maximum bending forces do not occur at the same time. Using a checkerboard loading with full dead load on all spans and live load on alternate spans produces the maximum end moment reactions on beams, whereas placing full dead and live load on all spans produces the maximum axial reactions on columns.

section 2306.

¹

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp. 165.

4.2.2 Wind Loads

Early skyscrapers were not vulnerable to lateral drift and oscillation caused by wind because of the heavy stone facades, small window openings, closely-spaced columns, massive framing members, and heavy partition walls that made the buildings extremely stiff and provided high damping. Furthermore, facade texture and many setbacks generated effective aerodynamic damping. In contrast, today's skyscrapers are light-weight structures that are much more flexible and thus more vulnerable to lateral sway and vibrations caused by turbulent winds.

It is extremely difficult to estimate the behavior of wind forces because they are not constant and static, but are dynamic and fluctuate in an unpredictable manner in magnitude and direction. Constant uniform wind pressures may be assumed for visualizing the lateral force action on a building, though it should be realized that the actual non-uniform wind pressure could generate torsion. However, this torsional effect may be treated as insignificant for symmetrical buildings for preliminary considerations.

Lateral wind loading normal to a building face is directly related to the wind velocity which consists of a constant average velocity and a varying gust velocity. As the air moves along the earth, it is retarded close to the surface due to drag. Thus, in the boundary layer, where buildings are located, the mean wind velocity increases with height. When the air flows passed a building, it causes a direct and positive pressure on one side and a suction on the other. The building must resist this total wind force. Typical wind pressure values range from 20 to 40 psf for ordinary high-rise buildings. More accurate values are defined in the codes and are based on Bernoulli's equations for steady streamline airflow. The code design wind pressure is:

$$P = C_e C_q q_z I, \quad 4.7$$

where C_e is the combined height, exposure, and gust factor coefficient, C_q is the pressure coefficient for the portion of the structure under consideration, q_z is the wind stagnation

pressure at the standard height of 33 feet, and I is the importance factor for the building.¹

Tornadoes are the most devastating winds with velocities as high as 500 mph. However, the probability that a particular building will be hit by a tornado is so small that structures are not usually designed for tornadoes; buildings designed for 200 mph winds are generally safe. This value should cover winds due to hurricanes as well; hurricanes occur in the US mostly along the Atlantic coast and typical design values are 90 mph for average hurricanes and 150 mph for strong ones.

The shape of the building has a substantial effect on the resulting wind pressures. For inclined or curved surfaces, the wind pressures may be taken as perpendicular to the plane projected vertically from the building face. For example, a building with a round plan has to resist only 60% of the wind load of a rectangular building with comparable dimensions. For a given wind direction, the streamlined tear-drop shape provides the least resistance to air flow.

As a building increases in height and slenderness, the dynamic action of wind does become a major concern. As the wind deflects around the building corners, vortices are generated alternately on one side of the building and then on the other, causing low-pressure areas which tend to pull the building towards them. This change in pressure occurs in a periodic manner depending on the wind speed and thus, causes the building to oscillate in the direction perpendicular to the wind. If these fluctuations act at intervals close to the building's natural period, the building will begin to resonate and the loads will build up drastically. This is a major concern for very tall and slender buildings; buildings with large slenderness ratios may be excited at lower wind speeds. Tapering of the building form and/or utilizing a mechanical damping system will reduce the vortex shedding.

The performance of a building may have to be investigated for wind action from different directions. Wind flow on more than one face causes double flexure and possibly torsion. Direct wind action is generally the greatest and controls the design. However, for an unusual building shape, it is extremely difficult to predict the most critical angle.

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"Uniform Building Code", International Conference of Building Officials, 1991, pp.154-155.

4.2.3 Seismic Loads

Like wind loads, seismic loads may often be treated as static lateral forces even though they are dynamic loads.¹ Unlike wind loads, which cause external lateral force actions on a building, earthquakes cause internal forces. Seismic forces are thus generated by the mass and stiffness of a building whereas wind forces depend on the exposed surface area. Hence, seismic loading is usually critical with respect to the performance of low- to mid-rise buildings while wind generally dominates for tall slender buildings.

Abrupt release of strain energy in an earthquake results in complex vibrations propagating at high speeds from the source in all directions through the earth and along its surface, reaching a given point on the surface at different times with different velocities and from different directions. The more significant seismic waves are the faster longitudinal P-waves which compress the earth in front and move a building's foundations back and forth in the direction of travel. The later arriving slower transverse S-waves oscillate in a plane perpendicular to the direction of propagation and tend to move a building's foundations up and down and side-to-side perpendicular to the P-wave. P- and S-waves travel through the earth. Q- and R-waves are relatively slow and travel along the surface. Since high-frequency components of seismic waves tend to weaken rapidly as they propagate away from the source, short-period low-rise massive buildings tend to be excited more near the epicenter while long-period tall buildings are excited more by the low-frequency waves that are transmitted over larger distances.

The primary effects of earthquakes causing possible building damage include ground rupture, ground failure, and ground shaking. Because of their random character, structural design for earthquakes can not be considered an exact engineering science; design is based

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Earthquakes also exhibit a vertical acceleration component. This vertical force action on buildings is usually neglected in design; there is no consideration for it in current building codes. However, in the recent events in Northridge, California, very large vertical accelerations were measured; the largest ever recorded! It is most probable that the building codes of the future will require some sort of consideration for vertical acceleration.

on assumptions rather than on precise data. Earthquakes are classified either according to their energy release or their intensity (destructiveness). The magnitude is measured according to the Richter Scale developed in 1935. The range is from 3 to 9, where each unit increase represents 32-times more energy release. An earthquake of magnitude greater than 6 is considered severe, 4 to 5 is moderate, and beyond 8 is a great earthquake. The intensity is measured by the Modified Mercalli (MMI) Scale developed in 1931. This scale is of a subjective nature with twelve degrees of intensity. The seismic risk map used by the Uniform Building Code (UBC) is correlated with the MMI scale. Buildings should resist minor earthquakes without damage, moderate earthquakes with some non-structural damage but no structural damage, and major earthquakes with some structural damage but no collapse.

For first approximations, a rough value for the seismic force may be assumed. As the ground suddenly accelerates in an earthquake, the inertia of the building mass tends to resist the movement. Assuming initially that the building is rigid and ignoring the effects of flexibility, structural type, and mass distribution, then the lateral inertial forces, from Newton's second law, are:

$$F = ma. \quad 4.8$$

Setting the weight of the building equal to $W = mg$ and a seismic coefficient of $C = a/g$,

$$F = WC, \quad 4.9$$

which shows that the magnitude of the seismic force is directly proportional to the weight of the building. It is common practice to express the seismic force as a percentage of the building weight. Typical values for high-rise buildings in major seismic zones range from 5% for flexible rigid frames to 20% for stiff bearing wall buildings. The building form and mass distribution determine the location of the resultant lateral seismic force. Furthermore, the form of the lateral force-resisting structure and its location within the building volume determine the type of action of the seismic force. For example, if the center of mass does not coincide with the center of resistance, twisting will be generated.

For analysis, the complex random ground vibrations may be visualized as known horizontal movements travelling back and forth. The building's inertial resistance causes the building to deform, activating its stiffness, and the random shaking of the base results in complex oscillations of the building. Modeling of typical building forms has shown that buildings may be treated as single-degree-of-freedom systems or as cantilevered pendulums with the mass lumped at the top. An increase in the height of the pendulum causes an increase in its flexibility which increases the natural period, T . However, this same assumption that the behavior of a multi-story building can be modeled by its fundamental period as a single-degree-of-freedom system is oversimplistic. Visualizing a high-rise structure as lumped masses at each floor along a vertical column produces a system with many degrees of freedom where the natural period is associated with each mode. However, for stiff multi-story buildings, it has been found that the fundamental period is dominant and that even for flexible buildings it contributes the largest influence.

Associating this with building response, a rigid building will move together with the ground without deflecting ($T = 0$) so that its acceleration is equal to the ground acceleration. A relatively stiff building with a natural period of approximately 0.3 s reaches a lateral acceleration much larger than the ground's, amplifying the ground motion along the building height. Flexible buildings with a natural period larger than approximately 1.4 s exhibit accelerations less than that of the ground's. Thus, tall flexible structures may be advantageous on sites where the ground motion tends to be of short period, though it must be considered that the later arriving long period waves may come close to the natural period of the structure. According to the UBC, flexible buildings with long natural periods attract less lateral force than stiff buildings with short periods.

The response spectrum of buildings are approximated by the UBC and defined mathematically by several coefficients. These coefficients account for several factors including the effect of soil conditions on the ground acceleration, structural and material behavior, seismic risk of certain zones, and importance of the building. The minimum lateral force for which a structure must be designed is given in terms of the equivalent base shear and is a function of the above mentioned factors:

$$V = \frac{ZIC}{R_w} W, \quad 4.10$$

where Z is the zone factor; I is the importance factor and takes into account that certain levels of damage control must be considered for certain situations, for example, facilities that provide essential services after an earthquake such as hospitals and fire stations; R_w is the structural system coefficient and C is a function of the soil coefficient and the natural period of the structure:

$$C = \frac{1.25S}{T^{2/3}}, \quad 4.11$$

S is the soil factor and represents the dynamic character of different site conditions. For example, sites with shallow, dense, stiff deposits tend to have short fundamental periods which are larger than the periods of the underlying bedrock, and sites with deep flexible soils tend to have long fundamental periods where the bedrock accelerations are amplified. On the basis of measurements made on existing structures, an approximation for T has been formulated:

$$T = C_f (h_n)^{3/4}, \quad 4.12$$

where C_f is a coefficient based on the framing system obtained from past performances of structural types, and h_n is the height of the structure.

The shear force at any level depends on the mass at that level and the amplitude of the oscillation, which may be assumed to vary linearly along the height of the building. The earthquake forces deflect the structure into its natural modes of vibration. As was mentioned above, though a high-rise structure is a multiple-degree-of-freedom system with many patterns of deformation, the first mode contributes the largest influence, especially for stiff or short-period buildings. For flexible buildings with longer periods, the higher modes of vibration indicate a whiplash effect which is taken into account by a concentrated load, F_t . F_t represents part of the total base shear, V , at the top of the building. The remainder of the load is distributed in a triangular fashion along the height of the building. F_t is present only when

$0.7 < T < 3.57$ and is given by:

$$F_t = 0.07TV < 0.25V. \quad 4.13$$

The remainder of the load, $V - F_t$, is distributed as concentrated loads at each floor level as follows:¹

$$F_x = \frac{(V - F_t)w_x h_x}{\sum w_i h_i}. \quad 4.14$$

The above method for equivalent static lateral forces is only applicable to regular building shapes with no vertical or horizontal irregularities in geometry, stiffness, strength, and mass. For irregular shapes, the distribution of the lateral forces must be determined considering the dynamic characteristics of the structure.

For design, the UBC requires that the lateral drift of one story relative to the adjacent story must not exceed 0.005 times the story height. At any floor level, the building must resist overturning caused by the governing lateral forces. In seismic zones 3 and 4, buildings taller than 160 feet must have ductile moment-resisting frames capable of resisting at least 25% of the required seismic force of the total structure. When designing for overturning caused by lateral loads, only the dead loads act to resist.

4.2.4 Load Combinations

Loads on a building that may act simultaneously should be combined. These loads are maximized when the probability of their combined action is less than their separate action and so the separate design loads may be reduced when they are combined. However, sometimes it is unreasonable to combine loads, for example the probability of a strong wind occurring

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"Uniform Building Code", International Conference of Building Officials, 1991, pp.158-164.

at the same time as a major earthquake is extremely small. Hence, the building codes do not require the design of structures for simultaneous action.

One common design philosophy is Allowable or Working Stress Design (ASD) in which a factor of safety to account for overloads, variations in material, etc. is placed on the yield stress of the material. Structural members are designed to carry the combined effects of the service loads to produce material stresses below the allowable stress. In some circumstances, load combination factors are used to reduce the combined effects of loads:

$$D$$

$$D + L + (S \text{ or } R)$$

$$D + (W \text{ or } E)$$

$$0.75(D + L + (S \text{ or } R) + (W \text{ or } E)),$$

whichever is greatest. ASD is the common practice for steel design.

The other commonly used design philosophy is Ultimate Strength Design (USD) or Load and Resistance Factor Design (LRFD), in which the factor of safety is placed on the loads rather than on the material. Structural members are designed to provide a load-carrying capacity exceeding that of the sum of the combined factored loads, referred to as the ultimate load. In steel design, the following factored loads are used:

$$U = 1.4D$$

$$U = 1.2D + 1.6L + 0.5(S \text{ or } R)$$

$$U = 1.2D + (0.5L \text{ or } 0.8W) + 1.6(S \text{ or } R)$$

$$U = 1.2D + 1.3W + 0.5L + 0.5(S \text{ or } R)$$

$$U = 1.2D + 1.5E + (0.5L \text{ or } 0.2S),$$

except for garages and areas of public assembly and where the live load is greater than 100 psf, where the load factor on L must be at least 1. For reinforced concrete design, the ACI building code recommends the following factored load:

$$U = 1.4D + 1.7L$$

$$U = 0.75(1.4D + 1.7L + 1.7(W \text{ or } 1.1E))$$

$$U = 0.9D + (1.3W \text{ or } 1.1E).$$

The design capacity of the structural member is a reduced value of the nominal strength of the member. The nominal strength is reduced by multiplying it by a resistance factor, ϕ , which is a function of the type of resistance induced in the member, such that the required strength is less than or equal to the design strength or:

$$R_u \leq \phi R_n$$

where u represents the combined effect of the factored loads and n represents the nominal load capacity. For members in axial tension, flexure, or both, $\phi = 0.9$; for shear and torsion, $\phi = 0.85$; for flexure in concrete, $\phi = 0.65$; for bearing in concrete, $\phi = 0.7$.

4.2.5 Dynamic Loads

Dynamic loads vary rapidly and generate vibrations, thus introducing the dimension of time. The dynamic properties of a building that are activated are mass, stiffness, and damping. Among the dynamic loads are impact loads which cause shock waves of short duration and longitudinal forces in the direction of movement on a structure. Dynamic load action causes larger forces to act on a structure than comparative static loads due to the shorter period of action. Such loads may be caused by wind, earthquake, oscillating machinery, moving elevators, blasting, or even from an impact of a car.

The dynamic properties of a building are measured by its natural period. Fundamental natural periods of typical buildings range from 0.1 s for single-story buildings, 0.6 s for 10-story bearing wall buildings, 1 s for flexible 10-story buildings, and 20 s for 20-story rigid frame building. While earthquakes apply sudden random forces with short periods, critical wind oscillations occur at longer periods. As was mentioned earlier, a state of partial

resonance may be approached for stiff buildings under seismic action and for flexible buildings under wind action. Thus, when the period of the source is much longer than the period of the building, the load may be treated as static and when the period of the source is shorter than that of the building, then the load must be considered dynamic with the appropriate increase in stresses.

4.2.6 Damping

For a given structure, the dynamic response can be reduced by increasing the stiffness, mass, or damping properties. Natural damping consists of internal damping which is a property of every building and is activated as the building deforms. There are three main types of internal damping:

- Hysteresis damping reflects the energy absorbed by the internal friction of the material in a stressed state.
- Viscous damping is produced by fluids such as air.
- Frictional damping arises from the friction between moving adjacent parts.

The extent of natural damping is on the order of 2 to 15% of the critical damping, which is the amount of damping required to stop oscillation before it completes one cycle. It has been found that tall concrete buildings have approximately 30% more damping capabilities than steel buildings.

Another form of built-in natural damping are base isolation systems. Base isolation controls the vibrations transmitted from the ground to the building by isolating the building from the ground and allowing it to float as a rigid body. Buildings on isolators hardly deform because the movement occurs at the isolators. For very tall buildings, natural damping with respect to wind may be achieved through aerodynamic damping. The shape of the building, texture of the facade, chamfered corners, openings, etc., cause wind turbulence which act as means for damping oscillations.

In addition to natural damping, artificial damping systems may be required. One form of artificial damping are passive dampers which are based on Coulomb friction, viscous friction, or shock absorber concepts. The active damping system that is currently in use is the tuned-mass damper. It consists of a large mass placed at the top of the building on a frictionless film. The mass is connected to the structure by a spring damping mechanism in the two major horizontal directions. The mass is tuned so that it can move at a period equal to the building's natural period. When the building sways, the mass tends to remain still as it compresses and pulls on the springs, which in turn tend to pull and push the building back on center. In other words, the tuned mass tends to oscillate with the same period as the building but in the opposite direction.

4.3 Force Flow

In a building, the horizontal and vertical structural planes must disperse the internal and external forces to the ground. Gravity load acting on a slab is transferred by the floor framing in bending to the vertical structural plane which may transfer the load axially to the ground. The type and pattern of force flow depends on the arrangement of the vertical structural planes. These planes may be vertical or inclined, continuous or staggered, may be evenly distributed or concentrated in the center or along the perimeter to form a core; the transmission of the loads may be short and direct or long and indirect.

From an efficiency point of view, the vertical loads should be carried along the shortest path possible to the foundation. However, sometimes it may be desirable to have optimum free ground space with a minimum of columns for high-rise buildings. For these conditions, the upper building mass must be linked to the ground using a special structural system. There are a great number of design possibilities available to achieve this. Among them are having the entire building facade act as a deep vierendeel wall beam spanning between columns, having a steel space frame carry the entire building to the base columns and/or a core, having the building blocks suspended from rigid frames which transfer the loads to

massive exterior columns or to a central core, or having the floors cantilevered from a central core. For preliminary design considerations, the upper and lower structural portions may be analyzed separately.

The horizontal floor planes not only transfer gravity loads to the vertical structure but also act as flat deep horizontal beams, or diaphragms, that carry lateral forces to the resisting vertical planes, which in turn act as vertical cantilevers to transfer the forces to the ground. Tall buildings respond to lateral forces primarily as flexural cantilevers in bending if the resisting structure consists of shear walls or braced frames, thus the behavior of these systems is controlled by rotation because of the high shear stiffness, and shear deformations may be neglected. A tall building will act as a shear cantilever when the resisting elements are rigid frames where the shear is resisted only by the beams and columns in bending. In this case, the effect of rotation is secondary and may be ignored in preliminary analysis.¹

The shape and location of the shear-resisting elements in a building have significant effects on their structural behavior under lateral loads. If the resultant of lateral loading acts through the centroid of a building, only translational reactions will be generated. However, if this force does not act through the center of rigidity, then twisting as well as translational reactions will develop. The twisting effects generate torsion in the structure. In a symmetrical building layout, the geometric center coincides with the center of mass and the center of rigidity, which are located at the intersection of the axes of symmetry. Torsion may also be generated in asymmetrical buildings where the center of mass does not coincide with the center of rigidity.

The effects of torsion cause many different responses. Simple torsion is resisted by shear stresses that vary with distance from the center of the section. The shear stress effect is essentially three-dimensional and produces complex responses including shear on mutually perpendicular planes and diagonal compression and tension effects. The several modes of failure resulting from torsion are: (figure 4.1)

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Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp.105.

- Transverse shear tends to cause failure by separation of adjacent cross-sectional surfaces (a).
- Longitudinal shear results in longitudinal splitting (b).
- Diagonal tension results in a spiral type of separation, which is especially sensitive for materials weak in tension (c).
- Diagonal compression results in spiral-type crushing of the material (d).¹

Figure 4.1 illustrates the general modes of failure of basic member sections under torsional effects. These torsional failure modes represent a simplified situation when applied to the larger scale of a "real" building structure composed of a more "open" cross-section or of discrete parts.

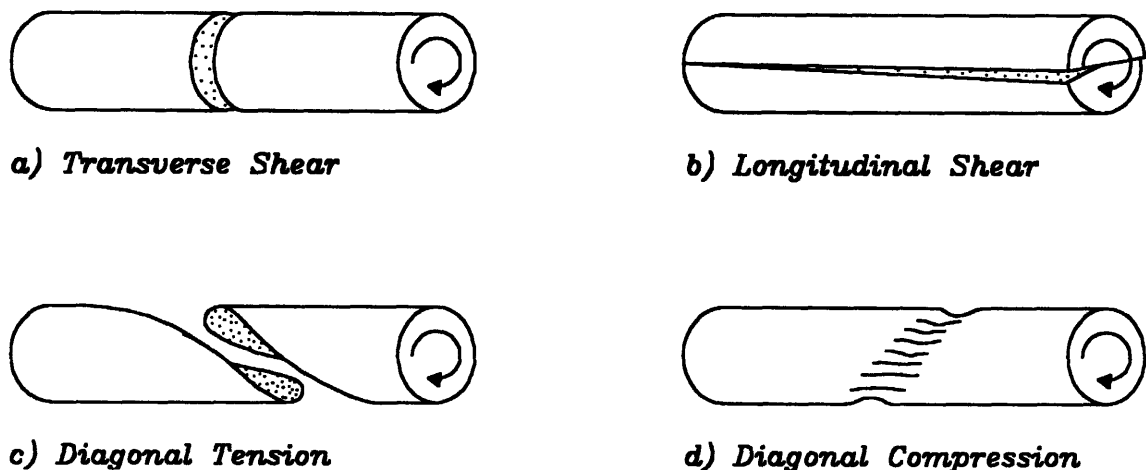


Figure 4.1: Failure modes due to torsion

Optimal torsional resistance is obtained with a closed core section which completely encloses a geometric shape. For a rectangular section, the translational forces are resisted by the walls parallel to the force. The torsion is resisted by the entire section where it is assumed that each wall represents a constant shear force because closed sections develop

¹

Ambrose, James, Building Structures, Second edition, John Wiley and Sons, Inc., 1993, pp. 78-79.

approximately uniform torsional stresses. The ideal shape for resisting torsion is a round closed thin-walled tube, where truly uniform shear stresses are developed. However, door, window, and other openings in a closed shaft will reduce its rigidity and thus its torsional resistance.

4.4 Behavior of Building Structures

4.4.1 Materials

On a structure, the magnitude of rotation due to lateral loads grows at a much faster rate towards the base than does the gravity load flow. The necessary reaction of a building to this force intensity distribution may be achieved in various ways, including tapering the building form, increasing the member sizes, or increasing the material strength towards the base. The logical form response of the building would be a shape defined by an exponential function. However, the more common and non-visual response of the structure is achieved by thickening of the members and/or increasing the material strength.

In steel construction, when strength controls the design, high-strength steels tend to be more economical than standard steels. However, because high-strength steels are more brittle, standard carbon steels may be more economical when stability, stiffness, or deflection control the design. The most common steel is A36 carbon steel with a yield strength of 36 ksi. High-strength low-alloy steels include A572 with a yield strength of 42 to 65 ksi, A441 with a yield strength of 40 to 50 ksi, and A588 with a yield strength of 50 ksi. A588 is a corrosion resistant weathering steel used primarily for exposed conditions. Thus, in high-rise buildings, high-strength steels are usually used at the bottom where dead loads constitute the major portion of the design loads, while lower strength steels with larger member sizes may be used in the upper portions where stiffness criteria govern.

In high-rise concrete buildings, reduction of column size in the lower portion of the building is essential to allow for use and flexibility of floor plan. Conventional concrete has

a compressive strength of 3000 to 5000 psi. The greatest strength concrete used to date in a conventional structure is a 19,000 psi concrete in Seattle's Union Square Building with a modulus of elasticity of 7.2×10^6 psi. It is interesting to note that the useful strength of concrete has been increasing from approximately one-tenth the strength of steel to nearly one-half at present.

4.4.2 Effect of Building Height

The structural design of ordinary buildings in the 20- to 30-story range is usually controlled by the gravity loading. In these cases, the gravity structure has sufficient strength to absorb the lateral forces. As the building height increases, the importance of lateral force action rises at an accelerated rate, the overturning moment being proportional to the square of the height. As the building becomes more slender, the lateral deformations become the primary concern, as the deflections increase with the height to the forth power. In the latter case, stiffness rather than strength becomes the controlling design determinant. For structural systems that utilize a central core as the sole support of all lateral loads, the core may have to be considered as a slender tower. The overall stiffness of a building depends on its height-to-width ratio, its form, and the structural system. The maximum lateral deflection of a slender tower is limited by some codes to $H/500$ for wind and $H/200$ for earthquake.^{1,2} These limits are placed for several reasons including architectural integrity, occupant comfort, and structural stability.

1

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp. 115.

2

"Uniform Building Code," International Conference of Building Officials, 1991, pp. 166.

4.4.3 Building Structure Response to Force Action

A building's structural system is expected to carry static and dynamic vertical loads, horizontal loads due to wind and earthquake, and resist and help in the damping of vibrations and fatigue effects. The most efficient structural system is the one that manages to combine all the structural sub-systems and components into a completely integrated system in which most of the elements take part in resisting the loads. These sub-systems and structural components are made up of the horizontal and vertical structural systems and the structural members that they consist of. The structural system as a whole should exhibit appropriate strength, enabling it to resist loads without collapsing, and stiffness, which enables it to limit deflections under loading.

The horizontal sub-systems act vertically to transfer the gravity loads in bending to the columns or walls, and act horizontally as a rigid horizontal diaphragms to transfer the horizontal loads to the lateral force resisting elements. The lateral force resisting structures act as vertical cantilevers fixed at the ground with concentrated horizontal loads acting at each floor level. The layout of the vertical sub-system may be symmetrical or asymmetrical and can range from a minimum of three planes to a maximum of cellular wall divisions and it may be located within the building as a core or along the perimeter of the building as a single unit or as separate planes. The vertical subsystem may also consist of framed members that are properly connected to resist the load actions. Walls are very rigid in their plane, especially when braced by rigid diaphragms. However, they offer very little resistance to forces normal to them. Shafts are stiff three-dimensional structures that carry the vertical loads tributary to them as well as act as good horizontal force resisting elements. By virtue of shell action, curved walls offer good lateral resistance, especially when braced by the flooring system.

A building under lateral loading will experience an overturning moment that is a function of the horizontal load magnitude and distribution. Assuming a resultant loading equal to H that acts at a location, a , along the building height, the moment, M , is equal to Ha . To achieve equilibrium of the building, the resultant of the resistance must become eccentric with the resultant of the building weight. In other words, $Ha = We$, where e is the eccentricity. If

the overturning stability is to be provided strictly by the weight of the building so that no uplift forces will be generated, e must fall within one-half the dimensions of the resisting structure. For symmetrical structures, this would be $d/2$, where d is the dimension at the base. Typically, e is taken as a value less than $d/4$ to $d/6$.¹

The bending on a building form due to the lateral loading causes compressive and tensile stresses on the cross-section of the resisting structure on either side of the neutral axis. The magnitude of these stresses is dependent on the distance the resisting material is from the neutral axis. Thus, the shape and the depth of the cross-section act together with the material properties to determine the overall resistance of the structure to bending. The most efficient shapes for resisting lateral bending and deflection are those that place the material away from the neutral axis in the direction of the force action. The vertical shear shaft is ideal for resisting overturning because it acts as a tube structure. As a whole, shafts are space structures that are usually significantly stiff and strong in any direction.

To avoid generating torsion on a building structure, there should be as much symmetry as possible, especially in regions where earthquakes are critical. Torsion is easily generated in asymmetrical buildings where the center of mass and the center of rigidity do not coincide. However, if a symmetrical building is loaded eccentrically, torsion will develop, though the effects will be small enough that they may be neglected for initial investigation. Torsion is most effectively resisted at points furthest away from the center of twist and by closed tubular shapes such as circles, squares, etc., because of their inherent torsional stiffness. Open sections, such as H , I , T , L , X shapes, are relatively weak in resisting torsion, especially the asymmetric shapes. In an open shaft, the twisting moment creates bending torsion with a non-uniform torsional shear. A closed tubular section, such as a shaft, is weakened by penetrations for door and window openings because uniform shear stress flow is disturbed around openings and warping will occur. When the openings occur in a vertical row, the core may be treated as an open tube, ignoring the coupling effect of the link beams. When the total area of penetrations is less than approximately 30% of the core wall

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Lin, T.Y., Stotesbury, S. D., Structural Concepts and Systems for Architects and Engineers, Second edition, Van Nostrand Reinhold, 1988.

area and the openings are small and arranged in a staggered fashion, then the effect of the openings may be ignored for preliminary design purposes and the core may be treated as a perforated tube.

The ideal shape for resisting torsion is a round closed thin-walled tube where only uniform torsional shear stresses are generated because of the complete symmetry. When a hollow shaft is twisted, it does not laterally displace or change its cross-section; under twisting, the original straight vertical lines form a helix so that the principal stress due to torsion form helices in compression and tension. These normal stresses can be transformed into pure shear at 45° to the inclined planes yielding shear stresses in the horizontal and vertical directions. For a circular core structure, the moment due to constant torsional shear flow, q , is:

$$T = qR(2\pi R). \quad 4.15$$

The torsional shear stress, f_{vt} , is the shear flow per unit area of the cross-section, or:

$$f_{vt} = \frac{q}{A} = \frac{T}{2tA_o}, \quad 4.16$$

where t is the thickness of the wall and A_o is the area enclosed by the centerline of the wall.

CHAPTER 5

Structural Components

5.1 Floor Systems

In tall buildings, the floor framing functions as a rigid diaphragm tying the building together and distributing the lateral forces to the vertical bracing elements as well as providing lateral support to them. The typical concrete slab construction for tall buildings may be considered as a rigid diaphragm. Under ordinary conditions, the slab can easily absorb the diaphragm stresses and is stabilized by the floor framing. The floor framing elements act in bending to transfer vertical loads to each other and then to the vertical supporting structure and act in compression or bending in the weak plane when transferring lateral loads.

Floor framing is composed of the primary beams or girders, secondary beams which span between girders, and joists or slabs that are supported on the beams. When the joist spacing is less than ten feet, a one-way slab system may be three to four inches thick and the joists may span relatively large lengths without being too deep. For heavier design applications or longer slab spans, the common system involves the use of metal decking laid on the steel joists or beams with the concrete poured onto it. The corrugated decking supplies the total bending strength while the concrete increases the slabs stiffness. Such decking commonly accommodates spans of five to ten feet between beams which rest on girders spaced 30 to 50 feet apart. If the slab is connected to the beams with shear studs, composite action may be achieved which may result in a decrease in depth of the system.

The depth of the floor structure depends on the span and the loads. Typical span-to-depth ratios for framing members are: 18 average with a maximum of 24 for steel beams spanning 15 to 60 feet, 14 average with 20 maximum for plate girders spanning 40 to 100 feet, and 25 for concrete on metal deck slabs spanning two to ten feet. For fixed end conditions, these values may be increased by 10% and for cantilevered members of typical spans, it is appropriate to use one-third of the values above.¹

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Lin, T. Y., Stotesbury, S. D., Structural Concepts and Systems for Architects and Engineers, Second edition, Van Nostrand Reinhold, 1988, pp. A9.

5.2 Trusses

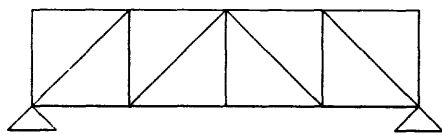
Trusses are very light two- or three-dimensional structural elements used as spanning systems. They act like long deep girders with the webs cut out. The top and bottom members of a truss are referred to as chord members and are analogous to the top and bottom flanges of a steel beam, and the members spanning between the chords are called web members. The repetitive modular units that comprise the web are referred to as truss panels. The panels are often made up of triangular units with pin-connections at the joints. Because of the natural rigidity of the triangle form, this system creates a rigid framework that can not be deformed. Typically, external loads are placed at the joints so that only compressive and tensile stresses are generated in the members. The overall moment on the truss is resisted by the top and bottom chords in tension and compression and the shear is resisted by the diagonal members in tension and compression. A Vierendeel truss is essentially a rigid frame in that the panels are rectangular in form and the joints are rigidly connected. The stability of a vierendeel truss requires the development of major shear and bending effects in the members and transfer of moments through the joints. The overall moment in the truss is resisted by the top and bottom chords in tension and compression and the overall shear is resisted by the chords in shear and bending. Each chord carries one-half the shear and the degree of frame action is determined by the stiffness of the connecting members; the stiffness of the vertical members should be at least four-times the chord stiffness for frame action to be achieved.

Some common truss configurations are: (figure 5.1)

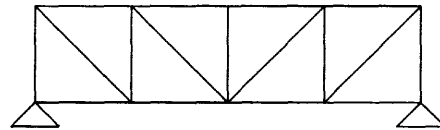
- The howe truss in which the tension members between the chords are all vertical and the compression members are inclined but not necessarily parallel to each other.
- In a pratt truss, the compression members between the chords are vertical and the tension members are inclined and parallel to each other.
- In a warren truss, all the chord members are of equal length, all the diagonal members are of equal length, and in each half of the truss, the diagonal compression members are parallel to each other and the diagonal tension members

are parallel to each other.

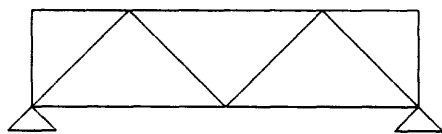
The major advantage of using a vierendeel truss is in its ability to accommodate penetrations without the interference of diagonal members. This is advantageous to architectural planning in many situations and in some cases, these advantages may be worth the additional cost for the heavier construction required.



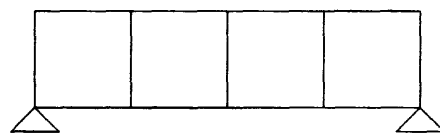
Howe Truss



Pratt Truss



Warren Truss



Vierendeel Truss

Figure 5.1: Trusses

The critical dimension of a truss is its depth. Typical span-to-depth ratios for common steel trusses are 12 average and 18 maximum.¹ Before analysis is performed, it may be desired to gain a sense for the forces that exist in the members of a truss. One way of accomplishing this is to visualize the probable deformed shape of the truss that would develop if a given member was removed. The nature of the force in that member could then be predicted on the basis of its role in preventing the deformation.² There are several methods of analyses for ordinary trusses, including the method of joints and the method of sections. Vierendeel trusses

¹

ibid.

²

Schodek, D., Structures, Second edition, Prentice Hall, Inc., 1992, pp. 116.

are statically indeterminate structures and so these simple methods of analysis can not be used on them. They must be analyzed by such methods as those used to analyze rigid frames.

For years, prestressing has proven to be very efficient in flexural and axially loaded concrete. Recently, tests on prestressing of steel trusses have also proven efficient. The prestressing is accomplished by applying a compressive force on the chord which under normal loading would be in tension. In other words, the truss is prestressed in such a way that the induced axial forces are opposite to those produced by normal loading. An existing prestressed truss system at the United Airlines hangar at O'Hare International Airport cantilevers 140 feet and the efficiency of the prestressing was proven by a 20% weight reduction as compared to a conventional design.¹

5.3 Vertical Sub-systems

Vertical elements of tall buildings have the primary purpose of supporting the gravity loads, thus acting in compression. However, they must also resist lateral loading transferred from floor diaphragms in shear and bending. Furthermore, the members must be capable of supporting the loads without excessive deflection.

Wall systems are very rigid in their plane and provide good resistance to lateral loads when properly braced by floors. They are relatively weak against horizontal loads applied normal to their plane because they have a large slenderness ratio in this direction. Common wall systems for tall buildings consist of reinforced concrete, steel trusses, masonry, etc. Rigid frame sub-systems consist of columns rigidly connected to beams or girders. The rigid connections cause the columns and beams to act in bending in resisting vertical and horizontal loads and to form a relatively stiff plane of overall resistance. Common framing systems are rigid steel frames, braced steel frames, and concrete framing.

Shafts are stiff three-dimensional structures that are usually made up of four solid or

¹

Melaragno, M., Simplified Truss Design, Van Nostrand Reinhold, 1981, pp. 106.

trussed walls. They carry vertical loads and are excellent in resisting lateral loads. When the shaft is relatively short and wide with a slenderness ratio under three, the dominant action is that of shear resistance. When the slenderness ratio is larger, bending requirements will most likely determine the design. For very large slenderness ratios, flexibility may be the controlling factor.

5.4 Design Concepts for Structural Members

The process of sizing structural members according to the codes is often complex and time consuming. In order to be able to concentrate on the building structure as a whole, simple processes and formulas for the preliminary sizing of elements have been developed. This is especially important at various times during design, for example when a sense for the forces and stresses must be obtained at the early design stage so that different structural schemes may be compared and evaluated.

In practice, the building design specifications of the AISC and ACI are the basis for structural design in steel and concrete, respectively. For the design of structural steel members, the traditional ASD approach is commonly used while the USD method is used for reinforced concrete.

5.4.1 Members Under Bending (*beams*)

The design of a beam is generally controlled by bending. Axial forces in beams are generally small and may be neglected for preliminary design. The flexural stress developed in a beam under a bending moment is:

$$f_b = M/S \leq F_b, \quad 5.1$$

where S is the section modulus given by I/c .

Steel beams

For a typical steel W-section, the allowable bending stress, F_b , is dependent upon the lateral support of the compression flange. For floors that are fully braced by the concrete slab, as is the case for most floors in tall buildings, $F_b = 0.66F_y$ for strong axis bending of doubly symmetrical members and is $0.75F_y$ for weak axis bending. For unbraced beams, $F_b = 0.6F_y$. Definitions of unbraced length values are found in tables in the AISC manual. Deflection limitation is generally based on live loading and is equal to $L/360$.

Composite action is common practice today in high-rise structures for spans larger than 25 to 30 feet. This system usually increases the strength and stiffness of the member by more than 50%. The bonding between the interface of the two materials is generally achieved by stud or channel connectors which resist horizontal shear. This allows a steel beam to act together with a concrete slab as a T-beam.

Steel plate girders are composed of relatively heavy flanges and relatively thin webs that are stiffened vertically by plates. For heavy loading conditions, a plate girder may provide a larger moment of inertia than that available for ordinary rolled shapes. The behavior of plate girders is in between that of rolled sections and trusses.

Concrete beams

In order to avoid complex deflection calculations for reinforced concrete beams, limiting span-to-depth ratios are available. The following approximate minimum depth values are commonly used (ACI sect. 9.5), where t is in inches and L is in feet¹:

Cantilever	$t = 3L/2$
Simple-span	$t = 3L/4$
Continuous-span	$t = 2L/3$

¹

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp. 252.

The above values are based on normal-weight concrete and grade 60 reinforcing steel.

Concrete members under bending are designed to resist the compression load of the force couple by action of an effective concrete area, or the rectangular stress block, and the tension load by action of the steel area, A_s . The dimensions of the compression stress block are b and a , where a is the depth of the stress block. The dimensions of the beam, b and h , are dependent on the amount of reinforcement used in the section as measured by the steel ratio $\rho = A_s/bd$, where d is the distance from the extreme compression fiber of the beam to the tension reinforcement, or the effective depth of the beam. Thus, the first step in designing a reinforced concrete beam is to assume a beam dimension and a steel ratio. The steel ratio must be selected within limits given by $\rho_{min} = 200/f_y$ and $\rho_{max} = 0.75\rho_b$, where ρ_b is given by:

$$\rho_b = \frac{0.85\beta_1 f'_c}{f_y} \frac{87,000}{87,000 + f_y} \quad 5.2$$

where $\beta_1 = 0.85$ for $f'_c \leq 4$ ksi or $\{0.85 - 0.05f'_c - 4\}$ for $f'_c > 4$ ksi, but shall not be less than 0.65. The product of bd^2 may be obtained from:

$$M_u = \phi \rho f_y b d^2 \left(1 - \frac{\rho f_y}{1.7f'_c}\right) \quad 5.3$$

where M_u is the factored moment. From the above equation, values of b and d may be assumed. Then a steel area may be determined from $A_s = \rho bd$. The final depth, h , of the beam may be determined by adding an adequate concrete cover to the steel, typically 1.5 to 3 inches.¹

When restrictions are placed on the dimensions of a cross section, the moment capacity of the section may be increased by adding additional reinforcing steel to both the compression and tension sides of the cross-section.

1

Leet, K., Reinforced Concrete Design, McGraw-Hill, Inc., 1982, pp. 71-91.

5.4.2 Members Under Compressive Axial Loading (columns)

The size and shape of a column depends both on magnitude and type of force action as well as its slenderness. As the slenderness of a compression member increases, its buckling capacity decreases. The degree of column slenderness is measured by KL/r , where K reflects the end conditions of the column, L is the unbraced length, KL is the effective length, and r is the radius of gyration about the weaker axis. Some typical values for K are 0.5 for both ends fixed, 0.7 for one end fixed and one end pinned, 1.0 for both ends pinned, and 2.0 for one end fixed and one end free to translate and rotate.

Steel Columns

A short steel column is one in which KL/r is approximately less than 40 and will probably fail in crushing or yielding of the material. A long slender column, with KL/r larger than 100, will most likely fail in buckling before the yield strength is reached. Intermediate columns will fail due to inelastic buckling. Typical building columns are short or in the intermediate range. Cross-bracing and truss web members generally exhibit long column behavior. According to AISC, the slenderness ratio for steel compression members is limited to 200. For a given column, the critical compressive axial load it can support is:

$$P_{cr} = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2 EA}{(KL/r)^2}. \quad 5.4$$

The allowable axial stress of a steel column member is based on KL/r and can be found in the AISC manual.

In addition to axial loads, most columns must also resist bending, especially in rigid framing systems. Axial compression reduces the capacity of a column in bending and visa versa. The proportioning of a steel column under axial and bending conditions may be accomplished by using the interaction equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1. \quad 5.5$$

where F_a is the allowable axial compressive stress based on the slenderness ratio assuming no bending, and F_b is the allowable bending stress as defined before assuming no compression.

Concrete columns

Because of the inherent continuity of cast-in-place concrete construction, concrete columns must always be treated as beam-columns. The primary moments due to member end moments cause deflections in the member which in turn creates secondary moments due to the axial load becoming eccentric from the centerline. The latter is referred to as the $P-\Delta$ effect. Concrete columns may be distinguished as short or long, where short columns are stiff and the secondary moments are small and long columns are slender such that the secondary moments may be significant. It should be noted that it is not the actual length of the column that determines its category, but rather the member's flexibility as a function of its length-to-thickness ratio and its end restraints.

According to ACI (section 10.11), a column may be classified as short when the effective slenderness ratio, KL/r , is less than or equal to 22 for unbraced columns. The compressive capacity of a short column consists of the strength of the concrete plus the strength of the steel, or:

$$P_u = \phi(0.85f'_c(A - A_s) + f_y A_{st}) \quad 5.6$$

5.4.3 Tension Members

In high-rise construction, tension members are found as the primary structural elements in trusses and suspension systems as well as in lateral bracing systems. When a member is strictly in vertical tension, cables or square or round rods may be used. Cable capacities may be more than five-times higher than that of milled steels, however, they are flexible and vulnerable to excessive elongation. When some rigidity is required to resist bending, such as may be the case for inclined or horizontal tension members, or the reversal of loading, rolled sections are more effective. Tensile members may be of absolute minimum

size due to the fact that no reduction of the allowable stresses are required for instability considerations. High-strength steels may be used with capacities of up to six-times higher than ordinary structural steels.

Most commonly, angles are selected for tension members. The preliminary design of tension members is based on yielding of the entire cross-section such that:

$$f_t = T/A \leq F_t = 0.6F_y . \quad 5.7$$

The elongation of a member under tension should be investigated and compensated for so that the supported elements will not become inclined. The elongation may be estimated by the following:

$$\Delta = \frac{TL}{AE} . \quad 5.8$$

Cables are often used as tension members, especially for vertical tension members. Cables are flexible members that consist of one or more groups of wires, strands, ropes, or bars. Several types of cables are available such as cables composed of parallel bars, wires, strands, or ropes, structural strands composed of wires helically coiled around a central wire, and parallel wire strands composed of individual wires arranged in a parallel configuration without the helical twist. Cables commonly have an ultimate tensile strength of 200 to 250 ksi, though they do exhibit a lower modulus of elasticity than rolled steel sections, thus flexibility and excessive elongations must be considered (refer to equation 5.5 above). The allowable tensile stress of a cable is dependent on the type of cable used. Some typical values are as follows:¹

1

Brockenbrough, R. L., Merritt, F. S., Structural Steel Designer's Handbook, Second edition, McGraw-Hill, Inc., 1994, pp. 14.33.

Cable type	Nominal tensile strength, F_U (ksi)	Allowable tensile strength, F_t (ksi)
Bars, ASTM A722 Type II	150	$0.45F_U = 67.5$
Locked-coil strand	210	$0.33F_U = 70$
Structural strand, ASTM A586	220	$0.33F_U = 73.3$
Structural rope, ASTM A603	220	$0.33F_U = 73.3$
Parallel wire	225	$0.40F_U = 90$
Parallel wire, ASTM A421	240	$0.45F_U = 108$
Parallel wire, ASTM A416	270	$0.45F_U = 121.5$

The required cross-sectional area of a cable may be found as follows:

$$A_{req.} = T/F_t \quad 5.9$$

CHAPTER 6

Structural Systems

A specific structural system derives its unique character from a number of considerations including: the specific functions of the structure, the geometric form or orientation, materials to be used, specific loading conditions, and usage of the final product. In attempting to find the ideal structure for a specific building, the designer must go through an exhaustive process of "comparative shopping".¹ Usually, there does not exist one ideal solution so a checklist must be constructed to rate the available systems for a given situation. Items to compare include:

- Economy of the structure and its construction.
- Special structural requirements: unique actions of the structure, development of the required strength and stability, adaptability to special loading, need for symmetry or modular arrangement.
- Design Problems: difficulty in performing analyses of structural behavior.
- Problems of construction: availability of materials, skilled labor, and equipment, and speed of erection.

With respect to gravity loads, the weight of a structure increases almost linearly with the number of stories. The weight of the vertical structural elements increase in this way because their weight is proportional to the axial stresses that determine their sizes. However, with an increase in building height and slenderness, the effect of lateral force action rises in a much faster non-linear fashion; the material needed for lateral force resistance increases with the square of the height. For typical structures of 20 to 30 stories, the vertical load resistance nearly offsets the effect of lateral loading; only about 10 percent of the total structural material is necessary for the lateral force resistance. At a certain height, the lateral sway of the building becomes critical so that stiffness rather than the strength of the material becomes the control in the design.

The efficiency of a given system is related to the quantity of material used. Therefore,

¹

Ambrose, J., Building Structures, Second edition, John Wiley and Sons, Inc., 1993, pp. 29.

optimization of structure for given spatial requirements yields maximum strength and stiffness with the least weight. This results in innovative structural systems applicable to certain height ranges. The various systems applicable to certain heights is only general because the many factors are not considered in such an oversimplified approach. Factors such as building shape, slenderness, functional requirements, etc, have an important bearing on the selection of an appropriate structural system.

6.1 Bearing Wall Structures

Bearing wall construction is typically used on building types that require frequent subdivision of space. Such buildings of 15 stories or more are common in brick, concrete block, precast concrete, or cast-in-place reinforced concrete, and have been built to heights of 26 stories. The dense, cellular arrangement of the walls of most bearing wall structures indicates the inherent compactness of such structures. In the most efficient layouts, the walls are oriented to provide lateral stability in the primary directions. The strength and stiffness depends on the degree of continuity between the horizontal floor planes and the vertical wall planes.

High-rise bearing wall buildings act primarily in compression. Because concrete is the most efficient material for compression, generally, bearing wall structures are of reinforced concrete. The load-bearing reinforced concrete walls may also serve as shear walls in resisting lateral forces in shear and bending. Especially when constructed of concrete, the self-weight of the load-bearing building is so large that it easily offsets the flexural tension due to lateral forces so that only gravity loads need be considered, at least for preliminary design purposes. To guarantee this, it is assumed in design that the stresses induced by lateral loads do not exceed one-third the compression caused by gravity loading.

Load-bearing systems may also be constructed of steel framing that is braced to resist lateral forces. However, when strength and rigidity is the controlling factor in design, a concrete structure is the most efficient one, though the steel structure may be more

advantageous when some flexibility of the structure is necessary, as in seismic areas.

6.2 Skeleton Buildings

Typical skeleton structures consist of parallel plane frames in the longitudinal and transverse directions. The frames may be organized in various ways. Continuous rigid frames in steel or reinforced concrete generally consist of a rectangular grid of horizontal beams and vertical columns connected by rigid joints. They resist lateral forces primarily in bending of the members, hence their efficiency is dependent on bay spans and member depths. Rigid frames are relatively flexible with respect to lateral loading so that lateral drift must be considered in design.

Hinged frames in steel or precast concrete consist of members that are pin-connected; only shear and axial forces are transferred between the members. Often, the hinged frame carries only gravity loading and is stabilized by rigid frames or other lateral force resisting systems. A few of these systems will be discussed in the following section on *braced frames*.

Semi-rigid connections allow some end rotation and thus develop partial end moments. These joints allow the redistribution of moments so that it is reasonable to design a joint for a moment equal to one-half of the full moment of a simply-supported connection. Ideally, rigid connections resist all rotation.

6.3 Braced Frame Structures

The concept of resisting lateral loads through bracing is the most common method applied to all building types. Though lateral bracing is often achieved through solid reinforced concrete shear walls or panels, the term "braced frame" generally refers to braced steel construction. Various bracing systems include rigid connections of beam-column joints, truss

frames, and diagonal bracing of pin connected rectangular panels. A plane frame may be braced along the full length or only in certain bays or portions of the building and may be stacked in a repetitive pattern or staggered.

The conventional concentrically braced frames are very stiff. In seismic regions, braced frames alone may not be sufficient to achieve acceptable structural behavior. To provide the minimum overall structural ductility during an earthquake, a secondary back-up ductile moment-resisting frame may be required, or the beam-column connections of the braced bays may be made moment-resistant so that ductile rigid frame action may take over should the diagonals become overstressed.

6.4 Frame and Shear Wall Interaction

At certain heights, generally a rigid frame structure becomes too flexible so that it must be stiffened by steel bracing, concrete shear walls, or some other method. The bracing usually occurs at the core of the building. However, a shear core at the center of a building becomes flexurally weak beyond a certain height, although it does provide shear resistance which reduces the shear deformations of the rigid frame. For tall buildings, the majority of the lateral loads are carried by the shear walls in the lower portion of the building and mostly by frame action at the top.

The interactive behavior of the rigid frame and shear wall structure depends on the relative stiffness of the components of the lateral force resisting structure. Usually, the shear walls provide most of the stiffness. For buildings of moderate height and for the bottom portion of tall buildings, the shear walls generally provide most of the lateral resistance.

6.5 Tube Structures

Load-bearing walls may be arranged to form a closed geometric section, or a tube structure, which can support all building loads, including the gravity and lateral loads. A tube is a three-dimensional hollow structure that is internally braced by rigid floor diaphragms and is most effective when it has a circular or nearly square cross-section. A cylindrical tube provides true tubular action and true three-dimensional response to lateral loads. A tube structure provides an efficient means for resisting overturning moment and shear in all directions and gives a building more strength and rigidity than from a shear wall or rigid frame system because the entire spatial structure acts as a unit. Furthermore, tubular sections are most efficient in resisting torsion. In analyzing a tube structure, it is assumed that the shear stresses are carried by the webs, or the walls parallel to the loading, and are uniform, and that bending is resisted evenly by the flanges, or the walls perpendicular to the loading.

The effectiveness of the behavior of a simple vertical hollow tube under lateral loading depends on the nature of the cross-section and the number of perforations in the web elements. A perforated tube is considered to have less than 30 percent perforations and may be assumed to behave as a perfect cantilever beam.^{1, 2} A framed tube has perforations of approximately 50 percent and can not develop tube action because of the flexible nature of its soft web elements. By increasing the depth of the frame beams, the lateral stiffness of the tube is greatly improved and the structure becomes a *deep spandrel tube*.

As the building height increase beyond approximately 60 stories, the slender internal core and the planar frames are no longer sufficient to effectively resist the lateral forces. At this point, the perimeter structure of the building must be activated to resist this action by acting as a huge cantilever beam. To accomplish this, the outer shell acts as a hollow three-dimensional structure or box-beam, where the external walls are braced by the horizontal rigid

1

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp.538-551.

2

Lin, T. Y., Stotesbury, S. D., Structural Concepts and Systems for Architects and Engineers, Second edition, Van Nostrand Reinhold, 1988, pp. 349.

floor diaphragms.

6.6 Core Structures

In many high-rise buildings, the lateral resistance of the structure is provided by one or more cores, which usually house the elevators, service ducts, and other vertical systems of the building. In some cases, the core, or cores, acts as the sole structural support for the building, carrying the gravity loads as well as resisting the lateral loads. The cores should be located, with respect to the plan layout, with as much a degree of symmetry as possible so as to create minimum eccentricity for lateral forces. Due to their geometry (closed geometric shapes), cores form natural tubes and so offer efficient means for resisting moments and shears in all directions, as well as good resistance to torsion.

As was mentioned previously, a core is weakened by openings for doors and windows. When the total area of penetrations is less than 30% of the wall area and the openings are small and arranged in a staggered pattern, the effect of the openings on the tubular behavior of the core may be ignored for preliminary consideration. When the openings occur in a vertical row and the spandrel beams are relatively slender, then the core must be treated as an open tube. The overall behavior of a core depends on its slenderness; the short massive core acts as a shear tube where the webs provide most of the resistance and bending may be neglected, whereas a core with a slenderness ratio more than 3 is controlled by bending and the flanges provide most of the resistance. For slender towers with a slenderness ratio greater than 7, flexibility becomes dominant in design.

When it is desired that free space be available at the base of the building, a core structure may be the ideal solution. This may be accomplished by cantilevering the floor framing from a central core. Because the core is solely supporting all vertical and horizontal loads, its bending stiffness is not as high as it would be had the core not been the sole support of the gravity loads. However, this effect does increase its shear resistance and stiffness because of the prestressed compressive state.

Another method for supporting the floors when an open space is desired at the base is on a massive base cantilever, where the floor loads are collected by story-high cantilevers at one or more levels and transferred to the core. The floors may also be suspended from story-high outriggers on tensile perimeter columns. This creates an excess load at the top of the core that is beneficial in that it prestresses the core from the top and keeps it under compression, thus increasing its stiffness.

6.7 Suspension Structures

This system offers an efficient use of material by employing hangers rather than columns to support floor loads. The application of the suspension principle to high-rise buildings began in the 1950's and 1960's. This structural system allowed for the minimum use of material and the expression of lightness of space and openness of facade by allowing for no visual obstruction from heavy structural elements. Suspending the floors on cables required only one-sixth of the material compared to using columns in compression.

To accomplish the suspension system from a single central core, floors are supported on vertical or diagonal tension members, commonly cables and rods or rolled shapes when some rigidity is required, which are hung from outriggers located at the top or at various heights along the core. As was mentioned above, the tensile members may be of absolute minimum size due to the fact that no reduction of the allowable stresses are required for instability considerations. High-strength steels may be used with capacities of up to six-times higher than ordinary structural steels.

CHAPTER 7

Construction Techniques and Their Effect on Design

The following is a *general* discussion on construction techniques and their effects on building design.

The construction of structures must be a vital consideration when dealing with structural concepts and systems. Building designs are often concerned only with the final building performance and usually do not place much emphasis on constructibility. It would be irresponsible of the designer to conceive of a scheme that looks good on paper but ignores the requirements for construction and the availability of trained labor and material resources. Construction of buildings involves a sophisticated process in terms of organization and scheduling flow of fabrication, shipping, and assembly, as well as of labor and equipment. The designer often lacks the experience to visualize how the contractor is going to construct the building. The designer must take the basic concepts of construction into account during the design stage so that problems of construction will not arise when the building process begins. The designer must appreciate the complexity of the construction process so that the specifications and drawings may be prepared to facilitate the building process.

In general, construction factors are not limited to work on the site but embody off-site issues such as fabrication of components, handling of materials, storage, transportation, as well as the on-site erection. The requirements of providing falsework when necessary to support the structure during construction and the availability of equipment required to accomplish the construction must also be considered. The greatest efficiency and economy of construction are achieved by using the minimum number of operations on site which includes minimizing the number of different components which must be assembled, simplifying the field connections, and minimizing the start-stop of activities; each trade should be able to perform its activity without any interference. The cost of material for a building structure can range from one-sixth to one-half the total cost of the structure with the remaining cost going to labor and equipment.¹ Therefore, in order to obtain economical construction, the designer must pay great attention to savings of equipment and labor. Dimensional organization of the building that reflecting simplicity and modular coordination of standard elements, including

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Lin, T. Y., Stotesbury, S. D., Structural Concepts and Systems for Architects and Engineers, Second edition, Van Nostrand Reinhold, 1988. p. 456.

the duplication of parts, greatly facilitates the construction process. For a regular building, this means uniform bay sizes and symmetrical organization of columns, and for a non-symmetrical building of complex configuration, breaking up the complexity into simple modular units. This repetition allows for a smooth flow of work where machines and other equipment may be utilized more efficiently. However, the designer must not allow this repetition of elements to determine the design and result in visual monotony.

7.1 Concrete Construction

Reinforced concrete construction may be advantageous because the concrete may be formed to any desired shape. Concrete may also be advantageous (over steel construction) when a heavier structure may be necessary as a stabilizing element against wind loading. Concrete construction may also allow for a reduced building height due to the floor systems consisting of flat slabs rather than slabs on steel beams. Concrete members are larger than steel members and thus provide more rigidity and better damping qualities, which may be preferable when stiffness is of concern. Finally, concrete construction does not require any special measures for fireproofing; the material in itself is fire-resistant.

Current technologies have made taller concrete buildings possible due to concrete's higher strength and so, have opened more options for designers in the selection of structural systems for high-rise buildings. At present, concrete buildings have been constructed to heights of 1000 feet, beyond which, however, the massiveness of the members may result in space inefficiencies.

Of the total cost of cast-in-place concrete construction, the concrete as a material is a relatively minor item, typically comprising approximately 15 to 20 percent, whereas the reinforcing steel constitutes a larger portion of approximately 20 to 30 percent. However, the bulk of the cost of concrete construction lies in the formwork, constituting up to 50 percent. This is due to the formwork constituting a structure in itself that must support the weight of wet concrete as well as other construction loads, and thus requires a great deal of attention in its design and construction. Furthermore, the erection and stripping of the formwork

requires a large amount of manhours. However, the cost of the formwork is inversely proportional to the number of times the forms can be reused. The mechanization of the work is very important to the economy and time of construction, and may be accomplished by making the forms reusable as is achieved through slipforming, flying forms, and lift slab construction.

For tall structures of constant or even tapered cross-section, slip forming may be used, in which forms are raised at a constant rate as concrete is poured. The forms are typically four feet deep and may be raised by electric, hydraulic, or manual jacks and the typical climbing rate is ten to twelve inches per hour, or one to two floors per day (24-hour period). This climbing rate allows for the concrete at the bottom of the form to have obtained a sufficient strength to accommodate its own weight as well as retain its shape. The slipforming assembly accommodates all of the disciplines required to form a concrete structure, with an upper deck from which the reinforcing and the concrete are placed and a lower deck from which the concrete is finished, if necessary, upon release from the form. A major advantage of the slipform technique, besides the speed of construction, is that the structure emerges with no construction joints which tend to be the weakest part of a concrete structure. Slip forming may not be the less expensive method of construction for a particular system, however the savings in time of construction will usually more than compensate for the higher cost of construction.

Designing for repetitive floor sections may allow for the use of flying forms, which are more economical than traditional formwork when used in 10 to 20 reuses. In this system, a prefabricated deck is hoisted in one piece from floor to floor as the slabs are formed. For consistent facade and framing systems, gang forms may be utilized, in which prefabricated modular panels are joined into larger units that may be folded and unfolded as they are moved from one vertical surface to form the next.

An alternative to minimize formwork at the site would be to utilize precast concrete members. Typically, tendons are pretensioned at the ends of the forming bed, reinforcing steel is placed in the bed and the concrete is poured. After the concrete has hardened, the tendons are cut, thus releasing the tensile force and causing the concrete to compress, producing pretensioned concrete. When planning and designing precast components, it is important to

consider the handling, transportation, erection, and connection problems involved with these components. The connections of precast concrete members often consists of wet joints, and when diaphragm action is necessary, the joints must be mechanically tied and a poured in place concrete topping must be placed. With precast concrete members, it is still possible to form any section, though it would be more economical to make use of commercially available units.

7.2 Steel Construction

Steel construction may be advantageous because it will result in a smaller building weight relative to concrete, which will result in lower foundation costs, and steel can provide the desired ductility in high seismic risk areas. With steel, the construction begins with shop-fabrication of the components. These components are then transported to the site, erected , and connected. Hardly any shoring or scaffolding is required with the exception of temporary lateral bracing. Because of this, steel buildings require less field labor than concrete buildings and the speed of construction is more rapid. However, steel does cost more than concrete and requires additional measures for fire protection. The fabrication and erection costs of steel are high, ranging from two- to four-times the cost of the material¹, thus ease of fabrication and erection is of prime significance to the economy of the construction. In order to reduce the fabrication costs, standard and locally available shapes should be used as much as possible, and these shapes should be combined into subsystems that can be easily erected and connected. Typically, two- or three-story column trees are fabricated at the shop to minimize the number of elements to be lifted and connected.

¹

Lin, T. Y., Stotesbury, S. D., Structural Concepts and Systems for Architects and Engineers, Second edition, Van Nostrand Reinhold, 1988, pp. 456.

PART 3

Precedent Buildings in Relation to the Spiral Building

CHAPTER 8

Core Structures

In order to accomplish the desired pure form of a spiral building rising into the sky like a spiral staircase climbing to the clouds, the obvious solution is to utilize a core structure so that the perimeter of the building will not be interrupted by vertical columns. The single central core can house all the vertical services the building will require including elevators, exit stairs, and mechanical and plumbing systems. Furthermore, this type of system will allow for the ground level to be free of structure and building mass, and, thus allow for open outdoor space shaded by the building above.

As was discussed earlier, there are several methods of attaching the floors to the core. Following, are some general discussions of these methods and examples of existing or planned buildings that utilize the systems. These existing buildings may be considered as precedent buildings in relation to the *Spiral Building*. For figures, refer to pages 75 through 79.

8.1 Core with Cantilevered Floor Systems

Cantilevered floor systems are not very common because of the flexibility of the floor structure and the relatively large moments generated at the support of the cantilever.¹ However, the proper design of this system may yield a strong and capable result. The structural strength of a single large concrete core cantilever structure is demonstrated by the 52-story *Singapore Treasury Building* which has 40 feet cantilever steel floor framing (figure 8.1). Another building that uses this system is the *Price Tower* in Bartlesville, Oklahoma, designed by Frank Lloyd Wright. This 16-story building uses four cross walls, or fins, that lie in two planes perpendicular to each other as the vertical support structure. The walls do not intersect each other so that the vertical services may be accommodated in the center of the building. The floor slabs are cantilevered off the fins at a 30° angle from the fins, except in one quadrant where the floors intersect the fins at right angles (figures 8.2 and 8.3).

1

Schueller, W., High-rise Building Structures, Second edition, John Wiley and Sons, Inc., 1986, pp. 86-88.

A structure of the mushroom type may consist of the floor loads being transferred to the core at only one or a few locations along the height of the core by story-high cantilevers. The 52-story *National Westminster Bank Building* in London has three massive cellular haunched cantilevers located at different heights along the core, each of which supports an office wing consisting of several floors (figure 8.4). Here, the floor loads are transferred to the core at only three locations. The core of this building consists of heavily reinforced concrete walls that were slip-formed at nearly four-feet thick. The 40-story *Rainier Bank Building* in Seattle, Washington, uses the pedestal principle in which the steel-framed office tower is supported on an outward sloping concrete pedestal base. Another building that uses this type of system is the 23-story *IBM Building* in Tel Aviv, Israel. Again, the floors surround the concrete core and are supported on a single haunched base. This base consists of reinforced concrete beams that are inclined upward from the core to support the perimeter columns and are braced from falling out by horizontal tension members (figure 8.5).

8.2 Central Core with Suspended Floor Systems

The system of suspending floors from tree-like central cores is quite common today. Giant arms are cantilevered from the top of the core to support the tensile columns at their ends that support the floors (figures 8.6). This system allows for a great reduction in the amount of material required because the members do not require a reduction in their allowable stresses due to their full cross-sectional area being utilized to support the tensile loads. Furthermore, the transferring of all the loads to the top of the structure prestresses the core and keeps it under compression, thus increasing its stiffness.

The 35-story *Standard Bank Center* in Johannesburg, South Africa, has massive story-high cantilevers from the central core at three levels, each supporting ten floors on eight prestressed concrete hangers. The floors of the 12-story *Westcoast Transmission Building* in Vancouver, Canada, are suspended by six sets of cables which are draped over the curved concrete core walls and two diagonally intersecting concrete arches of (figure 8.7 and 8.8).

Because the cables at the top are inclined, they experience additional components of stress and so are joined with additional cables to support the loads. Using this structural system saved 15% in structural costs over a conventional steel frame building plus 150 square feet of rentable floor space was added to each level. This type of structure is considered very effective in resisting earthquakes because it is allowed to move more freely so that less damage occurs.¹

¹

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp. 468.

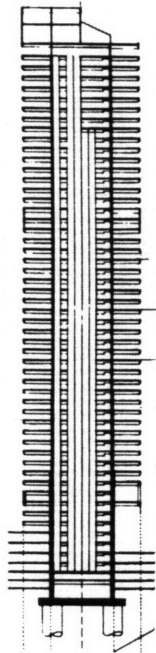
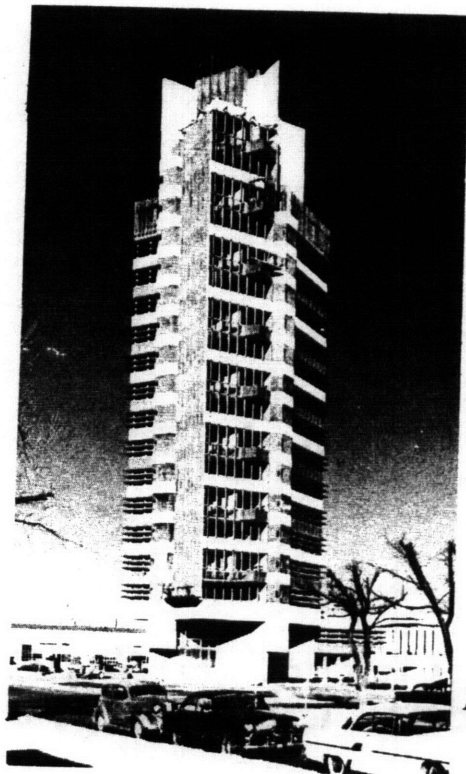


Figure 8.1: Singapore Treasury



*Figure 8.2: Price Tower
Building*

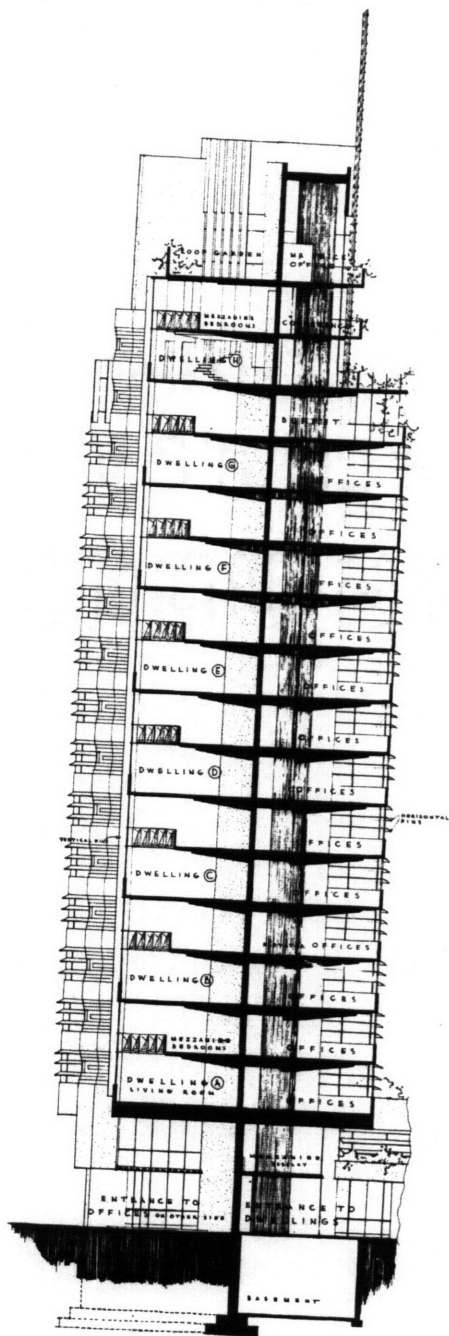
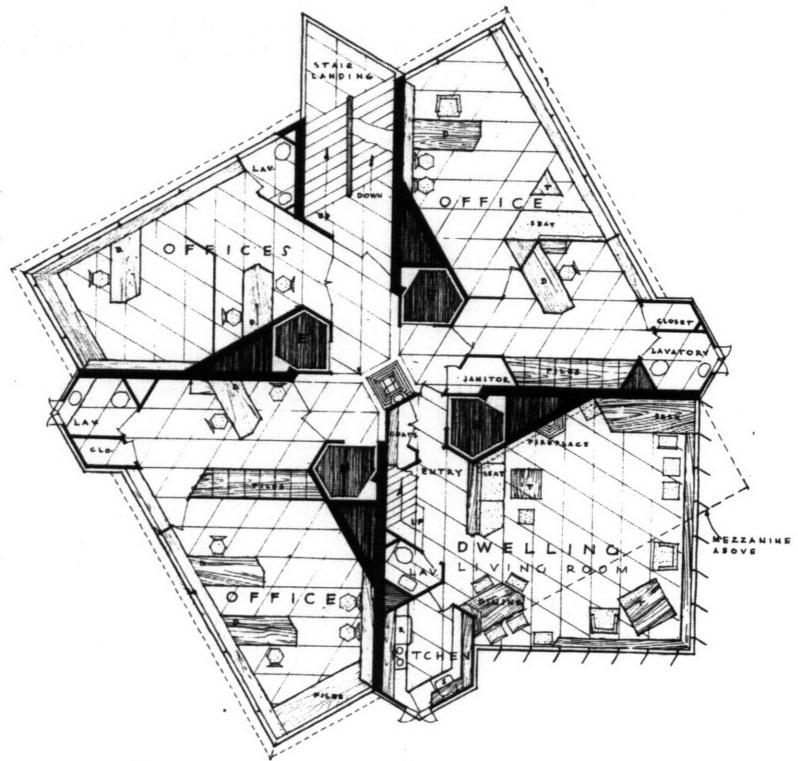
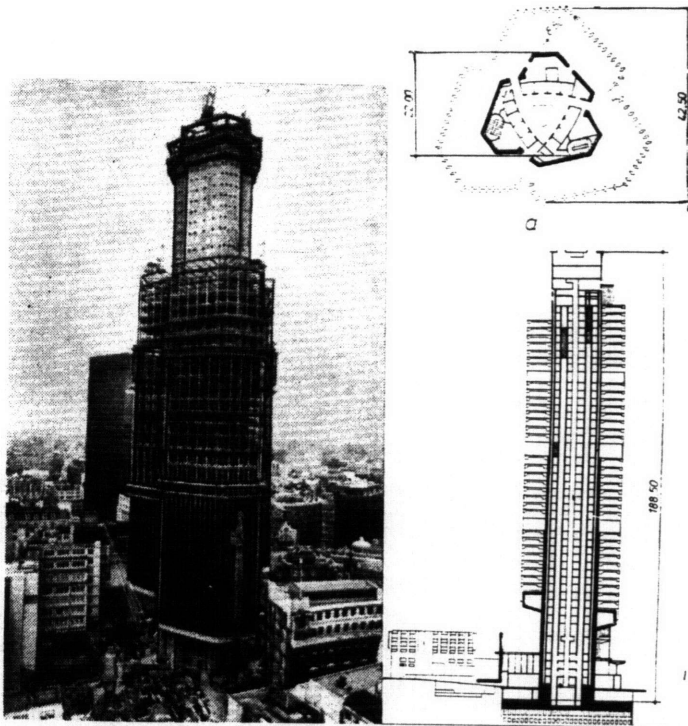


Figure 8.3: Price Tower





*Figure 8.4: National Westminster
Bank Building*

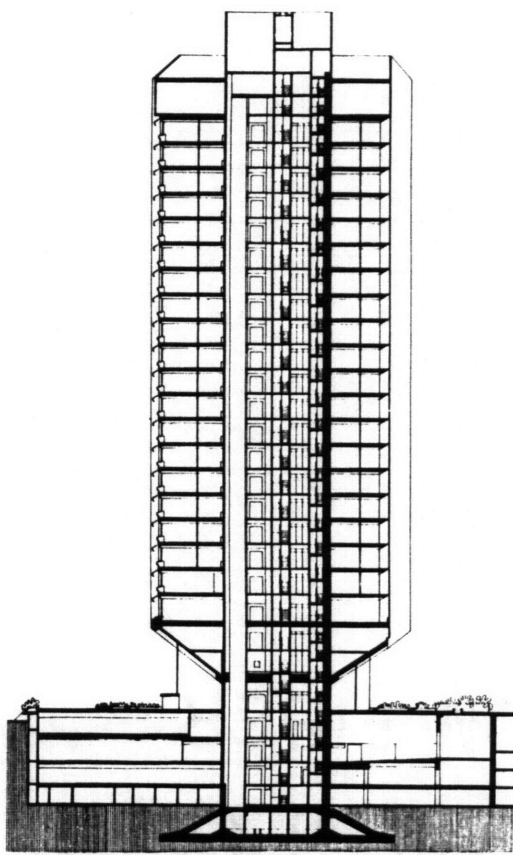
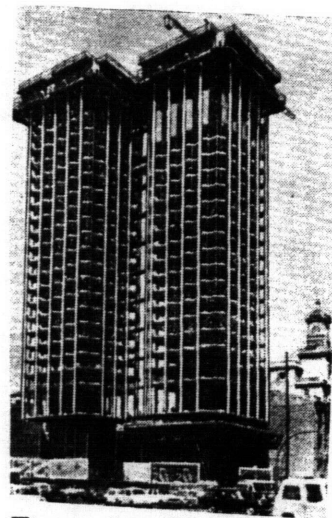
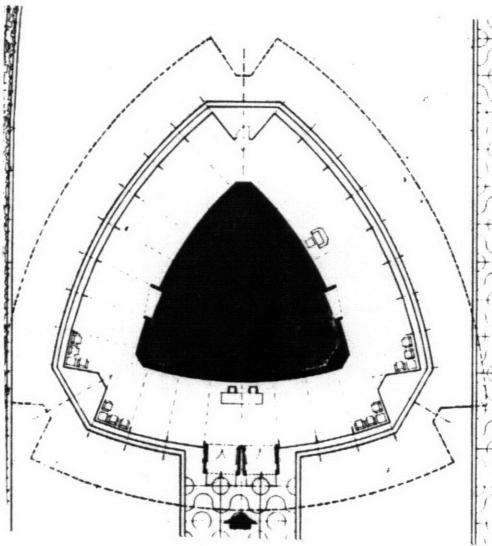


Figure 8.5: IBM Building



*Figure 8.6: Suspension
buildings*

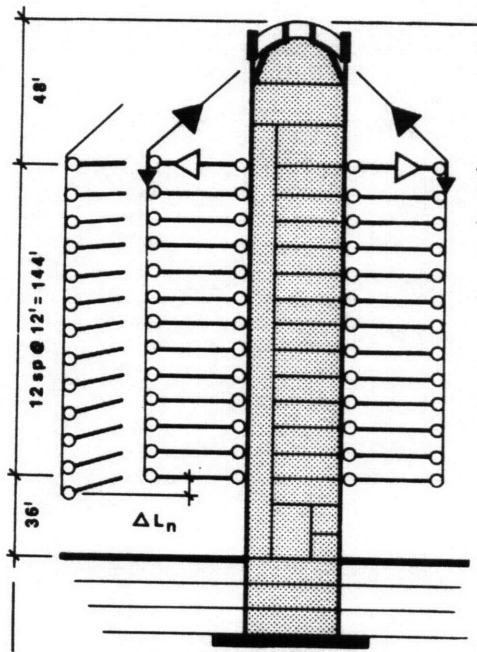


Figure 8.7: Westcoast Transmission
Building

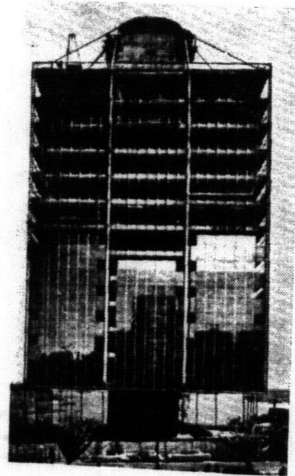


Figure 8.8: Westcoast Transmission
Building

CHAPTER 9

Current Spiral Building Forms

"The central city in many metropolitan areas separates people from nature. A reevaluation of the quality of life in the workplace is necessary and must lead to a new conception of the skyscraper that integrates the working environment with the natural environment."¹ In one particular spiral type design, the *Spiral Tower* by Roger Ferri and Welton Becket Associates of New York, nature was integrated through large garden atria at opposite sides of each floor. At successive floors, the outdoor terraces were offset by one bay moving counter-clockwise. Because the terraces were three floors high, each floor was able to look through the foliage of the preceding two floors as well as its own. This produced two great garden loggias on opposite sides of the building which were accessible from every floor. From the base of the building, these garden "stair cases" formed a pair of helixes that climbed the entire height of the shaft (figure 9.1).² This building was investigated only as a new type of tall building form and has not gone beyond that stage.

In Tel Aviv, Israel, there does exist an 8-story condominium building that takes the form of a spiral staircase. This building is known as *The Spiral* and was designed by Zvi Hecker. Each floor, consisting of one apartment each, represents one step of the staircase. Each step is rotated 22.5° creating open terraces on one end of the building and covered walkways on the other (figures 9.2 and 9.3). The concept of this building is very similar to that of the spiral building that is being investigated in this paper, the main difference being that Hecker's design allows for all floor loads to be directly transferred to the ground on bearing walls, whereas the floor loads of the spiral under investigation must first be transferred to a central core. In addition, Hecker's spiral represents a single "staircase" whereas four "staircases" are represented by the spiral under consideration in this paper.

It appears that the spiral building form is a common concept among Israeli architects.³ A design by Arie and Eldar Sharon for a spiral apartment building in Jerusalem, Israel, consists of apartment units cantilevered from a central core. In stepping the apartments in a

1

"Spiral Tower, NY", *Architecture and Urbanism*, Nov. 1985, #11, pp. 14.

2

"Spiral Tower, NY", *Architecture and Urbanism*, Nov. 1985, #11, pp. 14-15.

3

The author of this paper is of Israeli origin as well.

spiralling manner and locating windows only on the perimeter walls, a sense of privacy was created, as well as wide vistas and large outdoor terraces (figure 9.4). The form of this building is almost identical to that of the subject of this paper except that each floor level consists of three units rather than four.

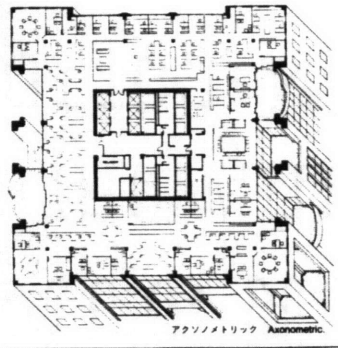
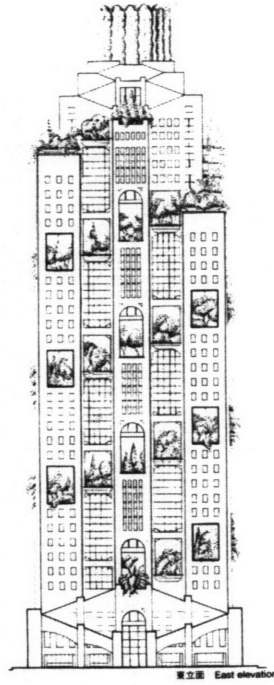
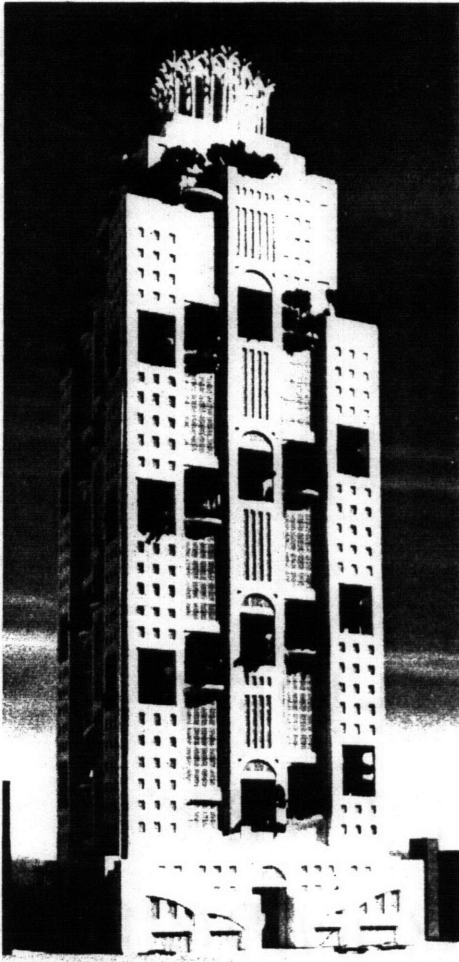


Figure 9.1: The Spiral Tower

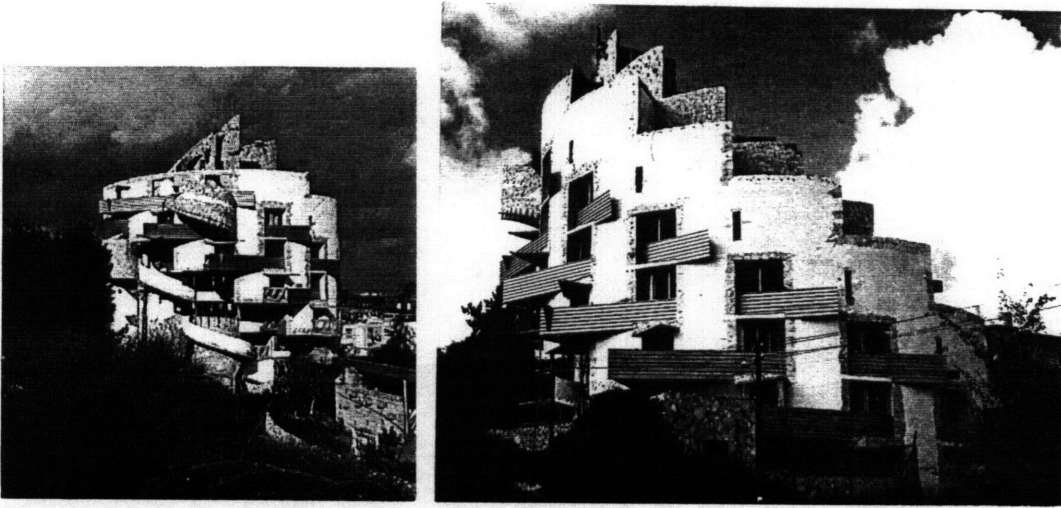


Figure 9.2: The Spiral - front and rear views

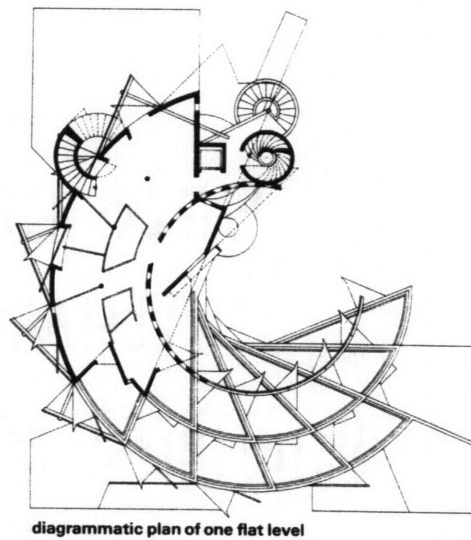
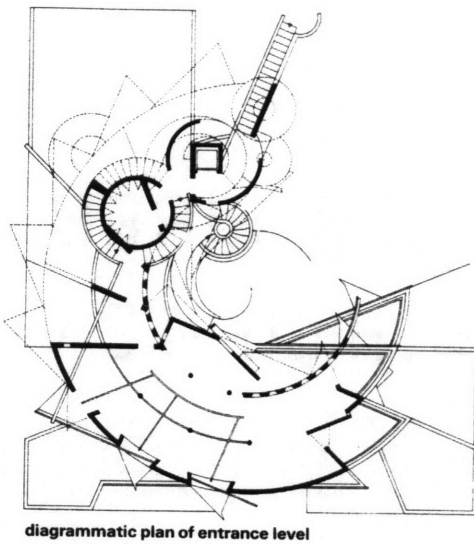


Figure 9.3: The Spiral - plan views

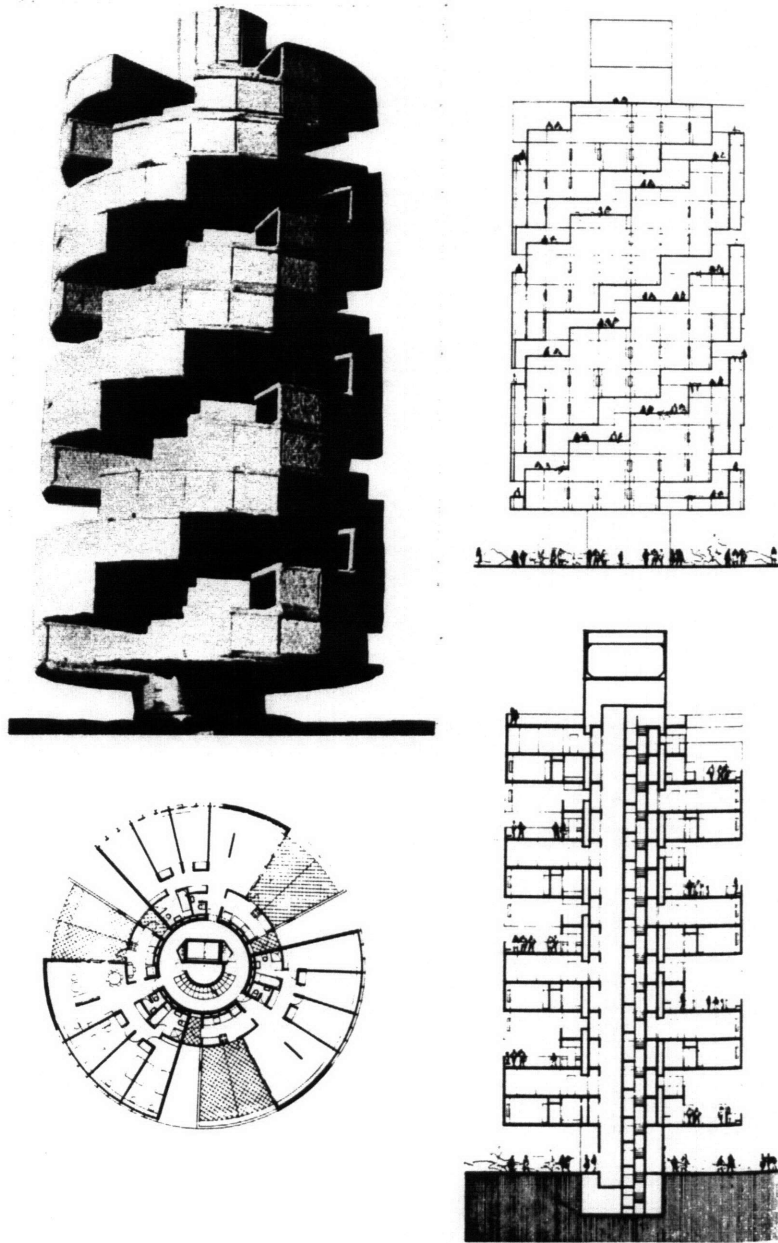


Figure 9.4: Spiral Apartment House

PART 4

Investigation of the Spiral Building

CHAPTER 10

Design of the Spiral Building

Now that the general process for designing a tall building has been discussed, it is possible to begin the investigation of the *Spiral Building*. In the following chapters, a step-by-step investigation will be performed so that a final building structure may ultimately be selected. First, determination of the loads, including gravity and lateral loads, acting on the structure and their flow path will be made. This will be followed by a brief discussion of some possible structural alternatives that may be applied to this problem, three of which will be selected for a preliminary investigation. The analysis of each of the three alternatives will allow for a comparison to be made so that an optimum and justified solution may be selected. Ultimately, a preliminary design for the *Spiral Building* will be obtained.

However, before beginning the investigation, some challenges involved in this specific problem shall be discussed.

10.1 Problems and Challenges in the Design

Due to its unusual form, the greatest challenge in the design of the *Spiral Building* will be in the design of the structural system. The spiralling shape may generate torsional forces induced by the gravity loads alone, as well as even larger torsional forces in the presence of lateral loading. This problem will require great stiffness in the overhanging floors as well as in the core. Furthermore, the effect of lateral loads, specifically wind loads, may be very difficult to determine due to the irregular profile of the building. A detailed wind analysis may not be possible without extensive wind tunnel testing and evaluation.

Another issue that must be considered is construction. The design solution must also provide a reasonable means of constructing the building, and this should be kept in mind during the design phase. Possible construction techniques will be discussed later in the comparison of the structural alternatives.

And, of course, there is the issue of cost. This final item is directly related to the above-mentioned issues; the more unconventional structure will cost more, as will a complicated and difficult construction process. It should be noted at this time that a cost

analysis will not be performed in this investigation, though it shall be stated that this building will not be cheap!

Upon completion of the structural analysis, a brief study of the building utility shall be performed. This will include space planning requirements, exit requirements, provisions for vertical and horizontal transportation systems including mechanical ducts, etc.

10.2 General Information

The following figures and values illustrate the information that will be necessary to perform the investigations of the *Spiral Building*. The values listed should be noted at this time for they may not be made reference to during the analyses.

10.2.1 General Building Dimensions

The following figures illustrate the general dimensions that will be used in the preliminary design of the *Spiral Building*.

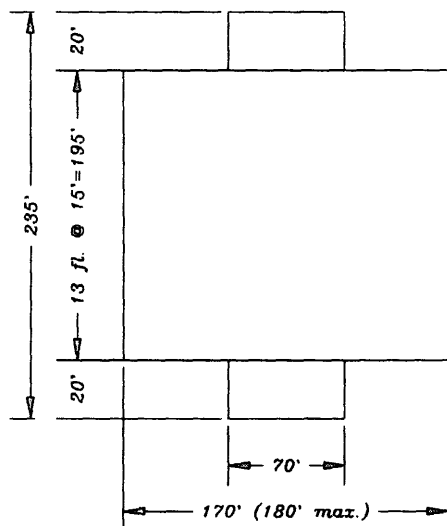


Figure 10.1: Building elevation

The diameter shown is the outside dimension of the core. The square service core is being assumed to be 900 square feet, though this may not be the final dimension.

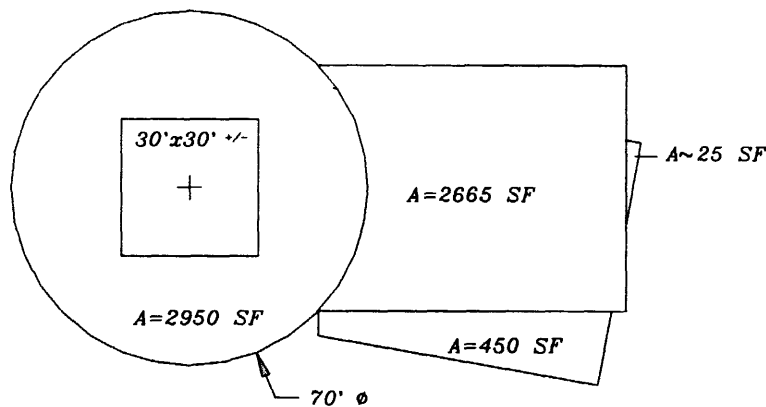


Figure 10.2: Plan layout

10.2.2 Material property values

The following values will be used throughout the analysis of the *Spiral Building*.

10.2.2.1 Steel properties

The design of steel members and systems shall be performed using the Allowable Stress Design (ASD) approach. Thus, the following values include the required factor of safety. All steel shall be A572 Grade 50 (for strength requirements) unless otherwise noted. All values are obtained from the AISC Manual for Steel Construction.

Yield strength	F_y	= 50 ksi
Modulus of elasticity	E	= 29,000 ksi
Steel weight		= 490 pcf
Bending strength (strong axis) F_{bx} (unbraced)	$= 0.60F_y$	= 30 ksi
F_{bx} (braced)	$= 0.66F_y$	= 33 ksi

Bending strength (weak axis) F_{by}	$= 0.75F_y$	$= 37.5 \text{ ksi}$
Tensile strength F_t	$= 0.60F_y$	$= 30 \text{ ksi}$
Shear strength F_v	$= 0.40F_y$	$= 20 \text{ ksi}$

10.2.2.2 Concrete properties

The design of concrete members and systems shall be performed using the Ultimate Strength Design (USD) method. Concrete shall be 8 ksi strength. The following values are obtained from the ACI Code.

Compressive strength f_c		$= 8 \text{ ksi}$
Modulus of elasticity E	$= 57\sqrt{f_c}$	$= 5100 \text{ ksi}$
Concrete weight		$= 150 \text{ pcf}$
Shear strength (pure shear) f_v	$= 2\sqrt{f_c}$	$= 0.18 \text{ ksi}$

Shear strength (shear/torsion + compression)

$$f_v = 0.002(1 + 0.0005 \frac{N_u}{A}) \sqrt{f_c}$$

Reduction factors

axial compression, with/without flexure	$\phi = 0.75$
shear and torsion	$\phi = 0.85$
bearing	$\phi = 0.70$

CHAPTER 11

Loads

11.1 Gravity Loads

The primary purpose of any structure is to resist gravity loads. The lateral force actions are secondary effects imposed onto the physical form. In order to begin any type of preliminary analysis, the dead and live loads that will act on the structure must first be determined. Dead load values are generally assumed from past experience and/or are taken to be the sum of the weights of the known items, such as structure, walls, flooring, mechanical ducts, ceilings, etc., that remain constant in the building over time. Live load values are generally obtained from building codes.

11.1.1 Determination of Dead and Live Loads Acting on the Building

The following exercise will consist of the determination of values primarily specified by the UBC. However, some dead load values must be assumed, specifically the weight of the floor framing system (not including the primary structural system). The floor framing will consist of beams at 10

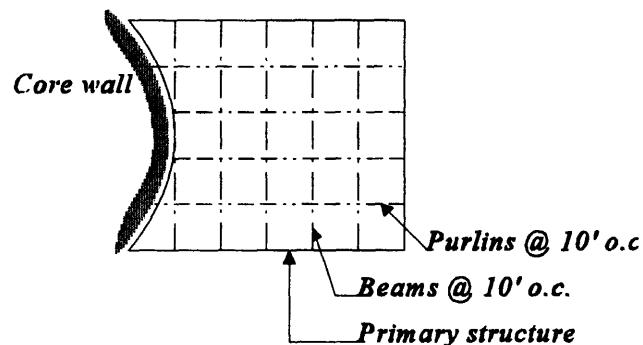


Figure 11.1: Floor framing plan

feet on-center spanning between the primary girders (~W24) and lateral support beams spanning between the beams at 10 feet on-center (~W10) (figure 11.1). Allowance is made for the placement of cross-members for the purpose of creating a rigid floor diaphragm. This will result in a structural weight of approximately 25 psf. Thus, for the preliminary investigation of the three chosen alternatives, a dead weight of 25 psf will be assumed for the floor framing. Note that this value is very conservative.

The live load values will be obtained from the UBC (1991 edition), tables 23-A and 23-C. The occupancies that will be under consideration for the *spiral building* include:

- Residential (#12)
- Cornices, marquees, and residential balconies (#4)
- Assembly areas (#3) (recreation room, lounge, and exercise room)
- Exit facilities (#5) (corridors)
- Printing plant - press rooms (#11) (mechanical room)
- Storage rooms (#18)
- Flat roof live loading (table 23-C, #1)

On members supporting areas of more than 150 square feet, the live load will be reduced by equation 4.6, which is not to exceed a value of 40% for members receiving load from only one level and 60% for other members.

The gravity loads acting on the various occupancies of the building are as follows. These values will be the same regardless of the structural system utilized.

Living units

- Dead loads (DL)

- | | |
|---------------------------|---------------|
| • Floor framing | 25 psf |
| • Flooring | |
| • concrete/metal deck | 50 psf |
| • finished floor | 5 psf |
| • Ceiling | |
| • mechanical/electrical | 5 psf |
| • insulation | 2 psf |
| • finished ceiling | 5 psf |
| • Interior walls (assume) | <u>18 psf</u> |

110 psf

- Total dead load, TDL

- Exterior walls - 20 psf of wall area over 210' x 10'
or 2100 square feet of wall area

42,000 lb.

- Live loads (LL)

- Living units (UBC, 23-A, #12)

40 psf

- Terraces (UBC, 23-A, #4)

60 psf

- Reduced live loads

- living units $R = 23.1(1 + 110/40) = 86.3\% > 40\%$

- $LL_{des} = 60\%(40) =$

24 psf

- terraces $R = 23.1(1 + 110/60) = 61.6\% > 40\%$

- $LL_{des} = 60\%(60) =$

36 psf

- Total loads (TL)

- Living units

- $TDL = 0.11$ ksf

- $TP = 42$ k

- $TLL = 0.040$ ksf

- $LL_{des} = 0.024$ ksf

$TL_{des} = 0.134$ ksf

$$TP = 42 \text{ k}$$

- Terraces

- $TDL = 0.110 \text{ ksf}$

- $TLL = 0.060 \text{ ksf}$

- $LL_{des} = 0.036 \text{ ksf}$

$$TL_{des} = 0.146 \text{ ksf}$$

Lobbies/corridors

The floor loads for the lobbies, as well as for the rest of the occupancies, will be slightly less than those for the living units. This is due to differences in flooring material as well as differences in the amount of wall partitions in each occupancy.

- Dead loads (DL)

- Floor framing 25 psf
- Flooring
 - concrete/metal deck 50 psf
 - finished floor 3 psf
- Ceiling
 - mechanical 5 psf
 - insulation 2 psf
 - finished ceiling 5 psf
- Total dead load, TDL 90 psf
- Walls - 20 psf of wall area over 2000 square feet 40,000 lb.

- Live loads (LL)

- (from UBC, 23-A, #5) 100 psf
- Reduced live loads
 - $R = 23.1(1 + 90/100) = 43.9\% > 40\%$
 - $LL_{des} = 60\%(100) =$ 60 psf

- Total loads (TL)

- $TDL = 0.090 \text{ ksf}$
- $TP = 40 \text{ k}$
- $TLL = 0.100 \text{ ksf}$
- $LL_{des} = 0.060 \text{ ksf}$

$$TL_{des} = 0.150 \text{ ksf}$$

$$TP = 40 \text{ k}$$

Miscellaneous

The following rooms, except the mechanical room, have, in addition to the calculated dead and live loads, a dead load of 42 k due to the weight of the outside skin.

- Mechanical room

- dead load 100 psf
- live load (UBC, 23-A, #11)
 - there is no reduction because $LL > 100 \text{ psf}$ 150 psf

$$TL_{des} = 0.250 \text{ ksf}$$

In addition, a concentrated load of 2.5 k must be considered in the design if the structural members (UBC).

- Storage room

- dead load 100 psf
- live load (UBC, 23-A, #18)
 - no reduction 175 psf

$$TL_{des} = 0.275 \text{ ksf}$$

$$TP = 42 \text{ k}$$

- Recreation room

- dead load 100 psf
- live load (UBC, 23-A, #3) 100 psf
 - $R = 23.1(1 + 100/100) = 46.2\% > 40\%$
 - $LL_{des} = 60\%(100) = 60 \text{ psf}$

$$TL_{des} = 0.160 \text{ ksf}$$

$$TP = 42 \text{ k}$$

- Exercise room

- dead load 100 psf
- live load (UBC, 23-A, #3) 125 psf
 - no reduction

$$TL_{des} = 0.225 \text{ ksf}$$

$$TP = 42 \text{ k}$$

- Lounge

- dead load 100 psf
- live load (UBC, 23-A, #3) 100 psf
 - $R = 23.1(1+100/100) = 46.2\% > 40\%$
 - $LL_{des} = 60\%(100) =$ 60 psf

$$TL_{des} = 0.160 \text{ ksf}$$

$$TP = 42 \text{ k}$$

Roof loads

- dead load (assume) 50 psf
- live load (UBC, 23-C, #1) 20 psf
 - $R = 23.1(1+50/20) = 80.8\% > 40\%$
 - $LL_{des} = 60\%(20) =$ 12 psf

$$TL_{des} = 0.072 \text{ ksf}$$

11.1.2 Total Building Weight

For preliminary investigations, the total dead weight of the building may be needed to determine a rough value for the seismic force as well as to determine rough dimensions of the core. The weight may also be used to determine the stability of the building. The following calculation of the dead weight does not include the weight of the primary structural systems (the core and the support systems to be investigated).

Living units

- individual

$$\begin{aligned} DW &= (0.134 \text{ ksf} \times 2665 \text{ sf}) + (0.146 \text{ ksf} \times 450 \text{ sf}) + 42 \text{ k} \\ &= 465 \text{ k/unit} \end{aligned}$$

- total contribution

$$TDW = 465 \text{ k/unit} \times (4 \text{ units/floor} \times 12 \text{ floors}) \quad 22,310 \text{ k}$$

Lobbies/corridors

- individual

$$\begin{aligned} DW &= (0.09 \text{ ksf} \times 2950 \text{ sf}) + 40 \text{ k} \\ &= 305.5 \text{ k/floor} \end{aligned}$$

- total contribution

$$TDW = 305.5 \text{ k/floor} \times 14 \text{ floors} \quad 4,277 \text{ k}$$

Mechanical room

- $TDW = 0.10 \text{ ksf} \times 3850 \text{ sf} \quad 385 \text{ k}$

Storage room

- $TDW = 0.10 \text{ ksf} \times 2665 \text{ sf} + 42 \text{ k} \quad 308.5 \text{ k}$

Recreation room

- $TDW = 0.10 \text{ ksf} \times 2665 \text{ sf} + 42 \text{ k} \quad 308.5 \text{ k}$

Exercise room

- $TDW = 0.10 \text{ ksf} \times 2665 \text{ sf} + 42 \text{ k}$ 308.5k

Lounge

- $TDW = 0.10 \text{ ksf} \times 2665 \text{ sf} + 42 \text{ k}$ 308.5k

Roofs

- $TDW = 0.05 \text{ ksf} \times (3850 \text{ sf} + (4 \text{ roofs} \times 2665 \text{ sf/roof}))$ 725.5 k

Total building weight, W

28,931.5k

11.2 Lateral Loads

As was mentioned in chapter one, the building will be assumed to be located on the Florida coast. The lateral force actions that are present in that area primarily consist of high winds; earthquake is not a concern. For the determination of the design wind loading, the following criteria, obtained from the UBC (p. 152-193), will be used (refer to equation 4.7):

- The exposure will be assumed to be "Exposure D - represents the most severe in areas with basic wind speeds of more than 80 mph and has terrain which is flat and unobstructed facing large bodies of water over one mile or more in width relative to any quadrant of the building site."¹
- The "fastest-mile wind speed", obtained from figure no. 23-1 of the UBC, is 100 mph, though the value of 200 mph will be used due to the threat of hurricanes.
- The value for C_e , obtained from table no. 23-G of the UBC, is 2.15.
- The value for C_p , obtained from table no. 23-H, #1, method 2, of the UBC, is 1.4 on vertical projected area and 0.7 on horizontal projected area.
- The value for q_s , based on a wind speed of 200 mph and obtained from table 23-F of the UBC, is 50 psf.
- The value for I , obtained from table no. 23-L of the UBC, is 1.0 for standard occupancy structures.

As was mentioned above, earthquakes in this region do not effect structural design. Florida is located in Zone 0, as can be seen from figure no. 23-2 of the UBC, which has a Zone Factor, Z , of 0 (refer to equation 4.10).

¹

"Uniform Building Code", International Conference of Building Officials, 1991, pp. 153.

11.2.1 Determination of Wind Load

For preliminary investigation, an estimated value for the wind pressure will be obtained as based on the vertical projected area of the building elevation; an area of 170 feet by 195 feet, or 33,150 square feet. The effects of corners and oblique angles caused by the rotation of the form will be neglected for preliminary investigation. Torsional effects due to wind may occur if the wind pressure is not consistent on either side of the building or when the wind encounters the corners and oblique angles of the building form. Because the torsional effects due to wind will be identical for the three structural schemes, these effects will not be investigated here. However, it should be noted that they will exist. From equation 4.7,

$$\begin{aligned} P &= C_e C_q q_z I \\ &= (2.15)(1.4)(50 \text{ psf})(1.0) = 150.5 \text{ psf} \end{aligned}$$

$$= 0.15 \text{ ksf}$$

The uniform load that will act along the building height is:

$$\begin{aligned} w &= P \times \text{building width} \\ &= (0.15 \text{ ksf})(170') \\ &= 25.6 \text{ k/foot} \end{aligned}$$

The resultant horizontal force will be:

$$\begin{aligned} H &= wh \\ &= (25.6 \text{ k/foot})(235') \end{aligned}$$

$$= 6012.5 \text{ k}$$

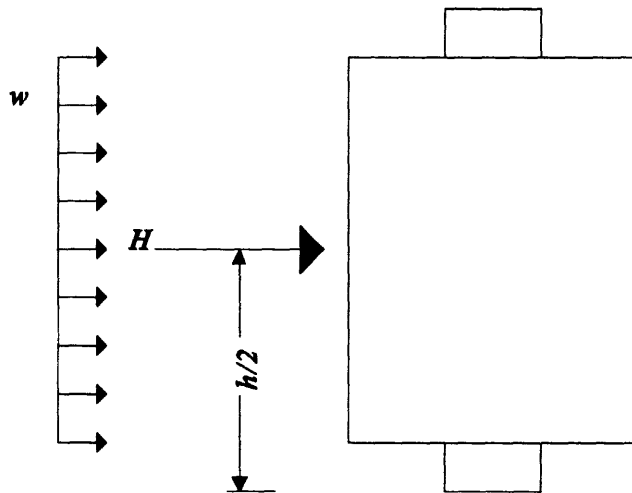


Figure 11.2: Wind loading

It shall be assumed that the resultant horizontal force will act at one-half the building height, though in reality, this may not be the case for this unusual building form.

11.3 Distribution of Loads and Structural Systems to Accommodate Them

The gravity loads must some how be transferred to the foundation from the central concrete core. First, however, the loads must be transferred from the floors to the core. This will be accomplished in bending by the floor system, either in cantilever bending as would be the case in a cantilevered floor system, or in simply-supported beam bending as would be the case in a suspension system.

The lateral loads will be transferred to the core by rigid diaphragm action of the floors. The floor system will be assumed to behave as a rigid diaphragm in transferring the lateral loads to the core, though the design will not be investigated in detail. For simplicity of illustration, the figures in the following sections do not specifically indicate such a structural system. Such a rigid diaphragm system may be accomplished by cross-bracing of the floor

framing, for example. However, given the general cruciform shape of each floor and a certain wind direction, one unit will transfer the load as a direct transverse force, while the two adjacent units will transfer the load in shear and bending. In either case, the floors must be rigid enough to transfer the loads without warping or tearing from the core.

There exists many possible structural solutions for this specific building problem. Following are just a few of the many possibilities:

- Steel framed floors cantilevered off rigid steel framed core.
- Steel framed floors cantilevered off reinforced concrete core.
- Story-deep steel trusses cantilevered off steel core.
- Story-deep steel trusses cantilevered off concrete core.
- Steel framed floors suspended from cantilevered trusses at top of concrete core.

Due to the core being the sole support of the gravity and lateral loading, and also that earthquakes do not pose a risk, the most efficient system for the core would be reinforced concrete, though it should be noted that a steel frame structure could be designed to accommodate the loading conditions investigated here. However, the design of the structure would be quite complex and the construction would be difficult, resulting in high cost and construction time. Reinforced concrete would be an advantage because it would lend itself to slipforming or climbing form construction, which would most likely prove to be the fastest and most economical systems and methods of construction for this building. For the flooring, steel framing would be most efficient because of the smaller amount, and thus weight, of material that will be required. Furthermore, because the floors will somehow be hung off the core, flexibility of the floor system may be a consideration in the design, in which case steel is the better option.

In the following chapters, three of the above-mentioned alternatives will be investigated on a preliminary basis and compared to determine which will be the optimum solution.

CHAPTER 12

Three Structural Alternatives for the Spiral Building

The following three alternatives will be investigated on a rough preliminary basis so that they may be evaluated individually and compared. This process will allow for a single optimum solution to be chosen.

Suspension System

The suspension system will consist of a reinforced concrete core and steel floor framing. The floor framing will be supported on tension members that will be suspended from story-deep trusses cantilevered at the top of the core. Each pair of trusses will be placed a distance of 20 feet on center from each other and will cantilever out nearly 60 feet from the core. The floor framing will be supported on the deep beams that will be supported by the tension members. These deep beams will be simply supported at the core wall and will span approximately 60 feet (figure 12.1). Transverse beams will rest on the deep beams and will cantilever out beyond the deep beams 15 feet on each side. The structural system will be discussed in more detail in the next section.

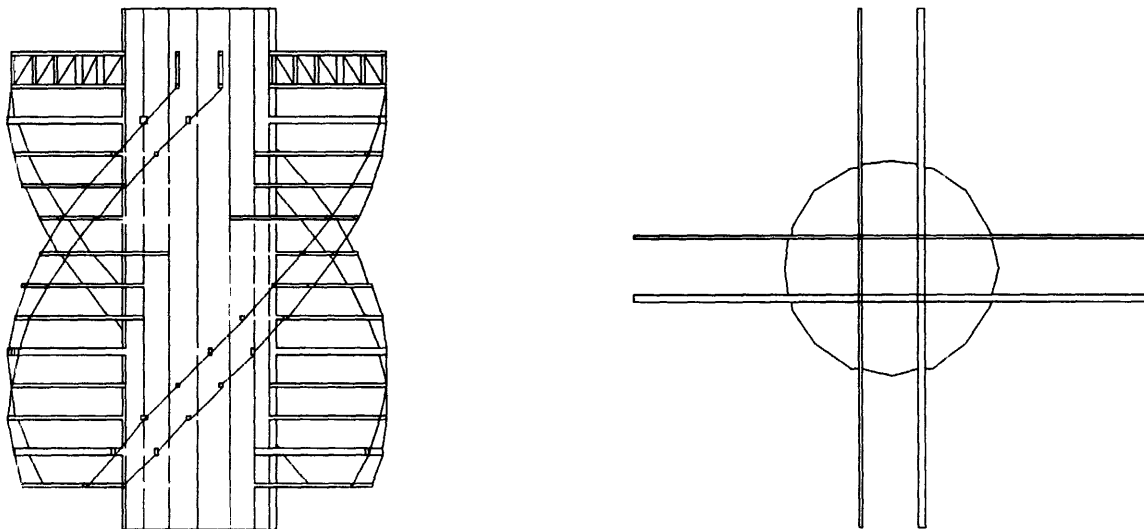


Figure 12.1: Suspension system

Story-height Vierendeel Trusses

This system will consist of story-deep Vierendeel trusses cantilevering off the core at every floor level. These trusses will be the primary support structure for the units. Each pair of trusses will be placed at 50 feet on center from each other. Each truss will be 170 feet long, being simply supported on the core and cantilevering out 60 feet from the core wall (figure 12.2). Deep beams spanning between the trusses at 10 feet on center will support the flooring of the units. These beams will cantilever beyond the trusses to support the roofing of the unit beneath and the terrace of the given unit.

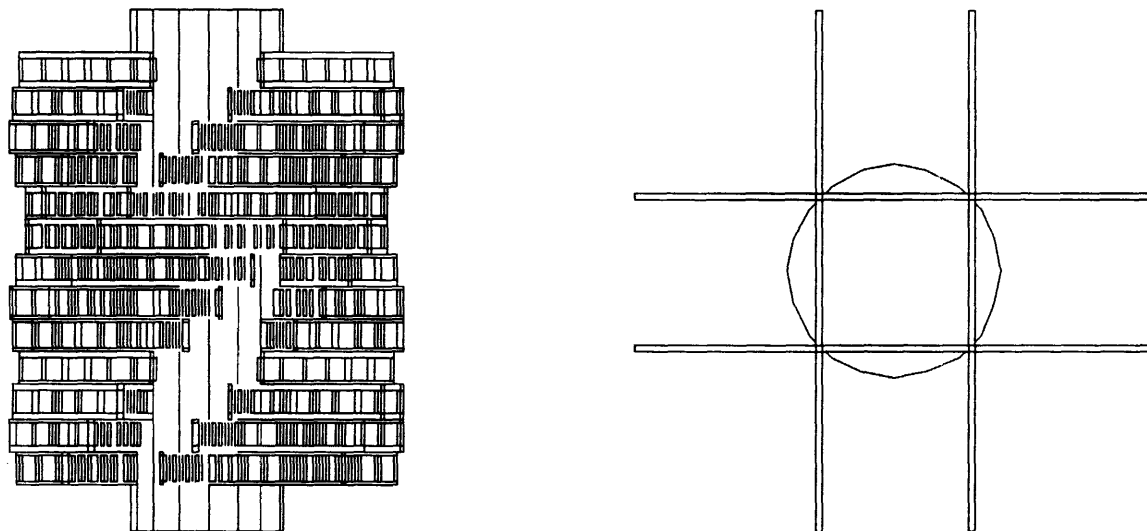


Figure 12.2: Story-height vierendeel trusses

Story-height Vierendeel Trusses on Alternating Floors

This system will be very similar to the one just described, however, the trusses will only be placed on alternating floors, specifically, on the odd-numbered levels (figure 12.3). The concepts will be the same as those above, however, the deep beams will now cantilever beyond the trusses to support both the roof of the unit below at one level and the floor of the

unit above at the upper level.

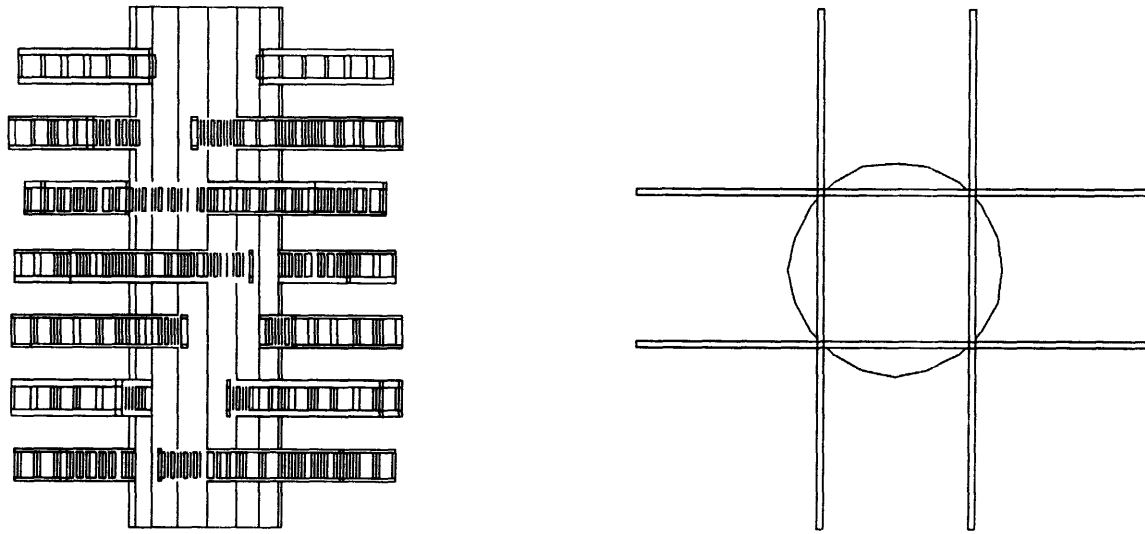


Figure 12.3: Story-height vierendeel trusses on alternating floors

In the following sections, each of the above three alternatives will be investigated on a preliminary basis so that an evaluation of each and a comparison of the three can be made to determine the optimum solution for the *Spiral Building*.

CHAPTER 13

Scheme 1: Suspension System

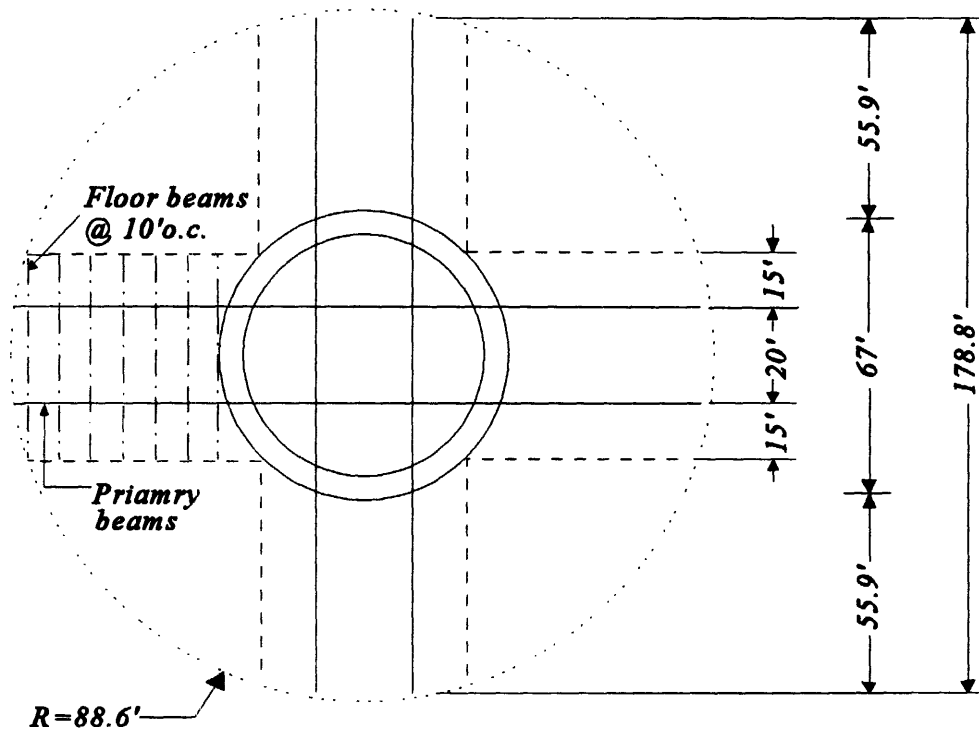


Figure 13.1: Plan dimensions of suspension structural system.

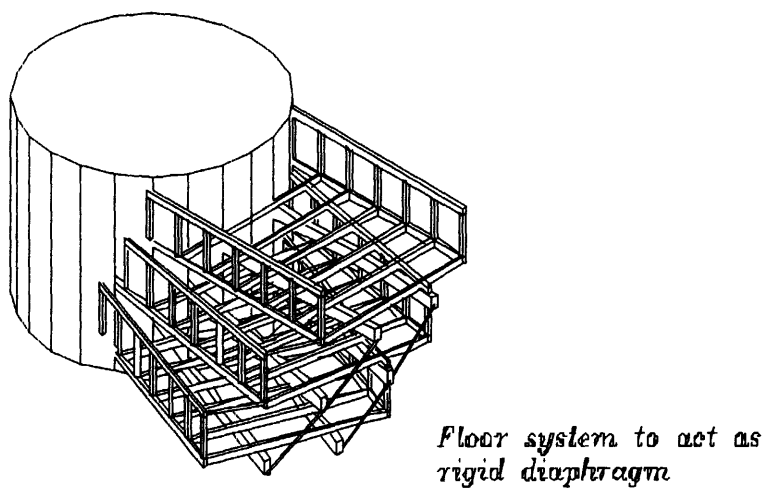


Figure 13.2: General view of suspension system.

13.1 Determination of Tributary Areas

Refer to figures 10.2 and 13.3 for the determination of the tributary areas.

- *Living units*

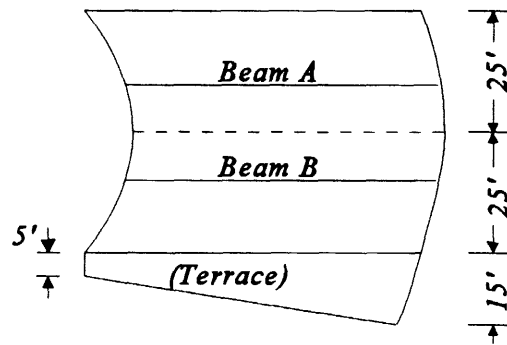


Figure 13.3: Tributary areas.

Total areas

$$\begin{aligned} A_{\text{unit}} &= 2665 \text{ sf} + 1/2(3.6')(50') \\ &= 2665 \text{ sf} + (\sim 100 \text{ sf}) \\ &= \underline{2765 \text{ sf}} \\ A_{\text{ter.}} &= \underline{450 \text{ sf}} \end{aligned}$$

Tributary areas

$$\begin{aligned} TA_A &= 1/2 A_{\text{unit}} = 1/2(2765 \text{ sf}) = 1382.5 \text{ sf} \\ TA_B &= 1/2 A_{\text{unit}} + A_{\text{ter.}} = 1/2(2765 \text{ sf}) + 450 \text{ sf} = 1832.5 \text{ sf} \end{aligned}$$

Tributary widths

$$\begin{aligned} tw_A &= 25' (@ 0.134 \text{ ksf}) \\ tw_B &= 25' (@ 0.134 \text{ ksf}) + \{5' \text{ to } 15' \rightarrow \text{Say } 12'\} (@ 0.146 \text{ ksf}) \end{aligned}$$

- *First floor rooms*

$$TA_A = TA_B = 1382.5 \text{ sf}$$

$$tw = 25' (@ \text{ appropriate load})$$

13.2 Gravity Loads on Beams A and B

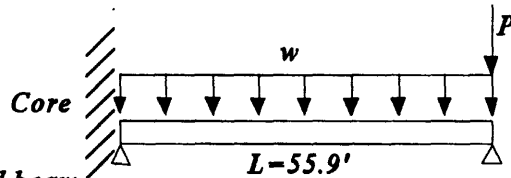


Figure 13.4: Typical beam.

- *Living units*
 - Beam A
 - loads acting on beam A (not including self-weight of beam)
 - distributed dead and live load of 0.134 ksf as calculated in section 11.1.1.
 - distributed dead load of $20 \text{ psf} \times 10' = 0.2 \text{ k}'$ from exterior wall in parallel direction.
 - concentrated dead load of $20 \text{ psf} \times 10' \times 25' = 5 \text{ k}$ at end from exterior wall in perpendicular direction.
 - total loads on beam A
$$w_A = (0.134 \text{ ksf} \times 25') + 0.2 \text{ k}' = \underline{3.55 \text{ k}'}$$
$$P_A = \underline{5 \text{ k}}$$
 - Beam B
 - loads acting on beam B (not including self-weight of beam)
 - distributed dead and live load of 0.134 ksf as calculated in section 11.1.1.
 - distributed dead and live load of 0.146 ksf as calculated in section 11.1.1.
 - distributed dead load of $20 \text{ psf} \times 10' = 0.2 \text{ k}'$ from exterior wall in parallel direction.

- concentrated dead load of $20 \text{ psf} \times 10' \times 25' = 5 \text{ k}$ at end from exterior wall in perpendicular direction.

- total loads on beam B

$$w_B = (0.134 \text{ ksf} \times 25') + (0.146 \text{ ksf} \times 12') = \underline{5.1 \text{ k'}}$$

$$P_B = \underline{5 \text{ k}}$$

- *Storage room*

- Beam A is equivalent to beam B

- loads acting on beams

- distributed dead and live load of 0.275 ksf as calculated in section 11.1.1.

- distributed dead load of $20 \text{ psf} \times 10' = 0.2 \text{ k'}$ from exterior wall in parallel direction.

- concentrated dead load of $20 \text{ psf} \times 10' \times 25' = 5 \text{ k}$ at end from exterior wall in perpendicular direction.

- total loads

$$w = (0.275 \text{ ksf} \times 25') + 0.2 \text{ k'} = \underline{7.08 \text{ k'}}$$

$$P = \underline{5 \text{ k}}$$

- *Exercise room*

- Beam A is equivalent to beam B

- loads acting on beams

- distributed dead and live load of 0.225 ksf as calculated in section 11.1.1.

- distributed dead load of $20 \text{ psf} \times 10' = 0.2 \text{ k'}$ from exterior wall in parallel direction.

- concentrated dead load of $20 \text{ psf} \times 10' \times 25' = 5 \text{ k}$ at end from exterior wall in perpendicular direction.

- total loads

$$w = (0.225 \text{ ksf} \times 25') + 0.2 \text{ k}' = \underline{5.83 \text{ k}'}$$

$$P = \underline{5 \text{ k}}$$

- *Recreation room and lounge*

- Beam A is equivalent to beam B

- loads acting on beams

- distributed dead and live load of 0.160 ksf as calculated in section 11.1.1.

- distributed dead load of 20 psf x 10' = 0.2 k' from exterior wall in parallel direction.

- concentrated dead load of 20 psf x 10' x 25' = 5 k at end from exterior wall in perpendicular direction.

- total loads

$$w = (0.160 \text{ ksf} \times 25') + 0.2 \text{ k}' = \underline{4.2 \text{ k}'}$$

$$P = \underline{5 \text{ k}}$$

13.3 Reactions from Dead and Live Loads

- *Living units*

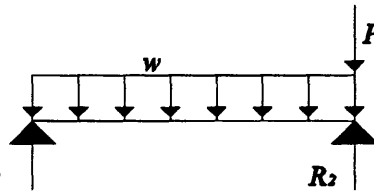


Figure 13.5: Beam reactions R_1

- Beam A - assume beam depth from a span-to-depth ratio of 20 (refer to section 5.1)

$$L/d = 20$$

$$d = L/20 = 55.9'/20 = 2.8'$$

- for a W-section of depth 3', estimate a beam weight of 0.35 k' (from AISC W-section tables)

- total load on beam A and reactions

$$w_A = 3.55 \text{ k}' + 0.35 \text{ k}' = \underline{3.9 \text{ k}'}$$

$$R_2 = P + 1/2wL$$

$$= 5 \text{ k} + 1/2(3.9 \text{ k}')(55.9') = \underline{114.2 \text{ k}}$$

$$R_1 = 1/2wL$$

$$= 1/2(3.9 \text{ k}')(55.9') = \underline{109.2 \text{ k}}$$

$$M_{\max} (@ \text{ center}) = wL^2/8$$

$$= (3.9 \text{ k}')(55.9')^2/8 = \underline{1528.8 \text{ 'k}}$$

- Beam B - similar to beam A, assume beam depth of 2.8' and weight of 0.35 k'
- total load on beam B and reactions

$$w_B = 5.1 \text{ k}' + 0.35 \text{ k}'$$

$$= \underline{5.45 \text{ k}'}$$

$$R_2 = 5 \text{ k} + 1/2(5.45 \text{ k}')(55.9') = \underline{157.33 \text{ k}}$$

$$R_1 = 1/2(5.45 \text{ k}')(55.9') = \underline{152.33 \text{ k}}$$

$$M_{\max} (@ \text{ center}) = wL^2/8$$

$$= (5.45 \text{ k}')(55.9')^2/8 = \underline{2128.78 \text{ 'k}}$$

For the rooms on the first floor, beams A and B are identical in their loading conditions and reactions. The same beam weights as those above will be assumed.

- *Storage room*

- total weight on beam and reactions

$$w = 7.08 \text{ k}' + 0.35 \text{ k}' = 7.43 \text{ k}'$$

$$P = 5 \text{ k}$$

$$R_2 = P + 1/2wL$$

$$= 5 \text{ k} + 1/2(7.43 \text{ k}')(55.9') = \underline{213.04 \text{ k}}$$

$$R_1 = 1/2wL$$

$$= 1/2(7.43 \text{ k}')(55.9') = \underline{208.04 \text{ k}}$$

$$M_{\max} = (7.43 \text{ k}')(55.9')^2/8 = \underline{2912.56 \text{ 'k}}$$

- *Exercise room*

- total weight on beam and reactions

$$w = 5.83 \text{ k}' + .35 \text{ k}' = 6.18 \text{ k}'$$

$$P = 5 \text{ k}$$

$$R_2 = 5 \text{ k} + 1/2(6.18 \text{ k}') (55.9') = \underline{178.04 \text{ k}}$$

$$R_1 = 1/2(6.18 \text{ k}') (55.9') = \underline{173.04 \text{ k}}$$

$$M_{\max} = (6.18 \text{ k}') (55.9')^2 / 8 = \underline{2422.56 \text{ 'k}}$$

- *Recreation room and lounge*

- total weight on beam and reactions

$$w = 4.2 \text{ k}' + 0.35 \text{ k}' = 4.55 \text{ k}'$$

$$P = 5 \text{ k}$$

$$R_2 = 5 \text{ k} + 1/2(4.55 \text{ k}') (55.9') = \underline{132.4 \text{ k}}$$

$$R_1 = 1/2(4.55 \text{ k}') (55.9') = \underline{127.4 \text{ k}}$$

$$M_{\max} = (4.55 \text{ k}') (55.9')^2 / 8 = \underline{1783.6 \text{ 'k}}$$

13.4 Sizing of Beams A and B

The member sizes for the primary support beams shall now be determined on the basis of bending (refer to equation 5.1), deflection, and shear. An allowable bending stress of $F_b = 0.6F_y = 30$ ksi will be assumed. The allowable deflection will be given by:

$$\Delta_a = L/240 = ((55.9')/240)(12"/') = 2.8".$$

- *Living units*

- Beam A

$$M_{\max} = 1528.8 \text{ 'k}$$

$$V_{\max} = R_2 = 114.2 \text{ k}$$

- required section modulus

$$\begin{aligned} S_{\text{req.}} &= M/F_b = ((1528.8 \text{ 'k})/(30 \text{ ksi}))(12"/') \\ &= \underline{611.52 \text{ in}^3} \end{aligned}$$

- select w-section from AISC, pp. 2-7 through 2-13

$$\text{W36x182 (S} = 623 \text{ in}^3\text{), I} = 11300 \text{ in}^4$$

$$\text{W18x311 (S} = 624 \text{ in}^3\text{)}$$

$$\text{W27x217 (S} = 624 \text{ in}^3\text{)}$$

$$\text{W24x250 (S} = 644 \text{ in}^3\text{)}$$

- select W36x182 (the reason for this selection is due to the probability that the other sections will most likely be inadequate in deflection requirements).

- check deflection

$$\Delta = \frac{5wL^4}{384EI}$$

$$= \frac{5(3.9k/')(55.9')^4}{384(29000ksi)(11300in^4)} \frac{(12in)^3}{ft^3}$$

$$= 2.63" < \Delta_a \rightarrow \text{OK}$$

- check shear capacity

$$f_v = \frac{V}{dt} = \frac{114.2k}{(36.33'')(0.725'')} = 4.33 \text{ ksi} < F_v \rightarrow \text{OK}$$

For beam A of units, use W36x182

- Beam B

$$M_{\max} = 2128.78 \text{ 'k}$$

$$V_{\max} = R_2 = 157.33 \text{ k}$$

- required section modulus

$$S_{\text{req.}} = M/F_b = ((2128.78 \text{ 'k})/(30 \text{ ksi}))(12\text{'})$$

$$= \underline{851.51 \text{ in}^3}$$

- select W-section from AISC, pp. 2-7 through 2-13

$$\text{W40x215 } (S = 858 \text{ in}^3)$$

$$\text{W36x260 } (S = 895 \text{ in}^3), I = 17300 \text{ in}^4$$

- select W36x260 so that depth will be consistent with that of beam A.
- check deflection (AISC, p. 2-296)

$$\Delta_{\max} = \frac{5wL^4}{384EI}$$

$$= \frac{5(5.1k/')(55.9')^4}{384(29000ksi)(17300in^4)} \frac{(12in)^3}{ft^3}$$

$$= 2.39" < \Delta_s \rightarrow \text{OK}$$

- check shear capacity

$$f_v = \frac{V}{dt} = \frac{157.33k}{(36.26in)(.84in)}$$

$$= 5.17" < F_v \rightarrow \text{OK}$$

For beam B, use W36x260

For the remainder of the rooms, it will be assumed that the shear has no effect on the beam design.

- *Storage room*

$$M_{\max} = 2912.56 \text{ 'k}$$

- required section modulus

$$S = M/F_b = (2912.56 \text{ 'k}) / (30 \text{ ksi})(12 \text{ '})$$

$$= \underline{1165 \text{ in}^3}$$

- select W-section

$$W24x450 (S = 1170 \text{ in}^3)$$

$$W27x407 (S = 1170 \text{ in}^3)$$

$$W40x297 (S = 1170 \text{ in}^3)$$

$$W36x328 (S = 1210 \text{ in}^3, I = 22500 \text{ in}^4)$$

- select W36x328 (the other sections are not available in the U.S.)
- check deflection

$$\begin{aligned}\Delta &= \frac{5wL^4}{384EI} \\ &= \frac{5(7.08k/ft)(55.9')^4}{384(29000ksi)(22500ft^4)} \frac{(12in)^3}{ft^3} \\ &= 2.9" \sim \Delta_s \rightarrow \text{OK}\end{aligned}$$

For storage room, use W36x328

- *Exercise room*

$$M_{\max} = 2422.56 \text{ 'k}$$

- required section modulus

$$\begin{aligned}S &= M/F_b = (2422.56 \text{ 'k}) / (30 \text{ ksi})(12 \text{ '}) \\ &= \underline{969 \text{ in}^3}\end{aligned}$$

- select W-section

$$\text{W27x336 } (S = 970 \text{ in}^3)$$

$$\text{W44x248 } (S = 983 \text{ in}^3)$$

$$\text{W36x280 } (S = 1030 \text{ in}^3), I = 18900 \text{ in}^4$$

- select W36x280 (the other sections are not available in the U.S.)
- check deflection

$$\Delta = \frac{5wL^4}{384EI}$$

$$= \frac{5(5.83k/')(55.9')^4}{384(29000ksi)(18900ft^4)} \frac{(12i)^3}{ft^3}$$

$$= 2.5" < \Delta_s \rightarrow \text{OK}$$

For exercise room, use W36x280

- *Recreation room and lounge*

$$M_{\max} = 1783.6 \text{ 'k}$$

- required section modulus

$$S = M/F_b = (1783.6 \text{ 'k})/(30 \text{ ksi})(12"/') \\ = \underline{713.44 \text{ in}^3}$$

- select W-section

$$\text{W24x279 } (S = 718 \text{ in}^3)$$

$$\text{W36x210 } (S = 719 \text{ in}^3), I = 13200 \text{ in}^4$$

- select W36x210

- check deflection

$$\Delta = \frac{5wL^4}{384EI}$$

$$= \frac{5(4.2k/')(55.9')^4}{384(29000ksi)(13200ft^4)} \frac{(12i)^3}{ft^3}$$

$$= 2.63" < \Delta_s \rightarrow \text{OK}$$

For recreation room and lounge, use W36x210

13.5 Tension Members

The tension members will have axial forces in them greater than the vertical reactions calculated previously (R_2) due to their inclination from the vertical position. This inclination will also result in a horizontal force component that will tend to cause the deep beams, A and B, to rotate about their joints at the core wall. Bracing the deep beams against each other and utilizing the rigid floor diaphragm principle will allow the entire floor system to act as a unit in bending in resisting this rotation. Basically, the floor will act as a deep cantilever beam in resisting the moment, with each of the deep beams, A and B, acting as the beam flanges in tension and compression. For the investigation, it shall be assumed that the floor system is rigid enough so that warping of the shape will not occur.

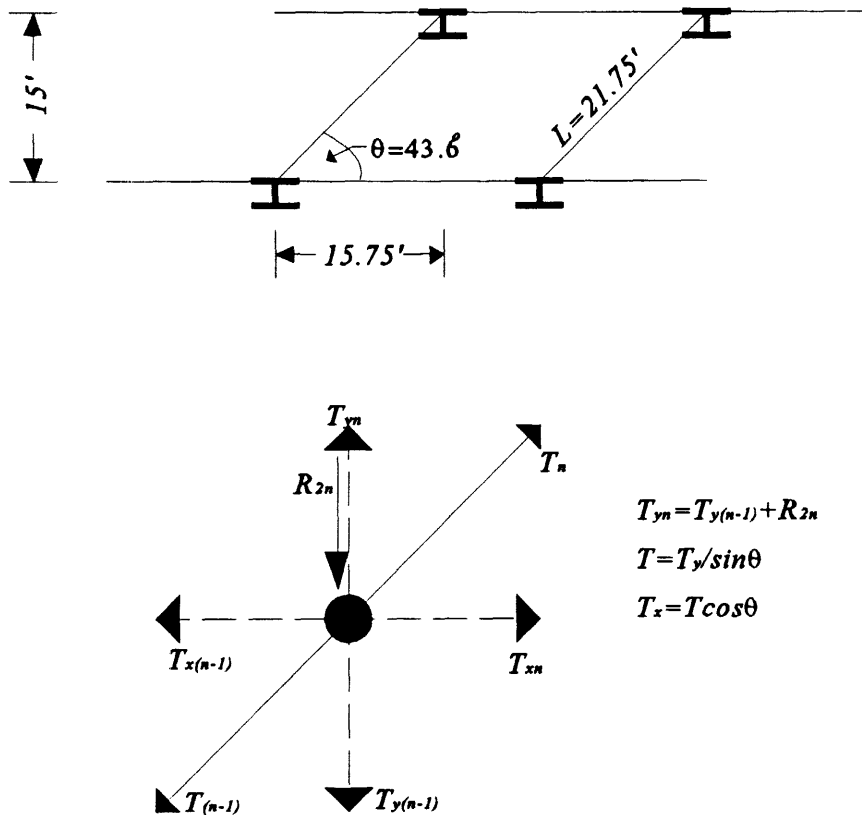


Figure 13.6: Tension member forces.

13.5.1 Determination of Forces in Tension Members

The following analysis will be performed for beam B. Refer to figure 13.6 for the directions of the forces.

- *Begin analysis at first floor (storage room)*

$$R_2 = 213.04 \text{ k}$$

$$T_{y1} = R_2 = 213.04 \text{ k}$$

$$T_1 = T_{y1}/\sin\theta = (213.04 \text{ k})/(\sin 43.6^\circ) = 308.9 \text{ k}$$

$$T_{x1} = T_1\cos\theta = (308.9 \text{ k})(\cos 43.6^\circ) = 223.7 \text{ k}$$

- *Second floor*

$$R_2 = 157.3 \text{ k}$$

$$T_{y2} = T_1\sin\theta + R_2 = (308.9 \text{ k})\sin\theta + 157.3 \text{ k} = 370.3 \text{ k}$$

$$T_2 = T_{y2}/\sin\theta = (370.3 \text{ k})/(\sin 43.6^\circ) = 536.9 \text{ k}$$

$$T_{x2} = T_2\cos\theta = (536.9 \text{ k})(\cos 43.6^\circ) = 388.8 \text{ k}$$

- *Third floor*

$$R_2 = 157.3 \text{ k}$$

$$T_{y3} = T_2\sin\theta + R_2 = (370.3 \text{ k})\sin\theta + 157.3 \text{ k} = 525.6 \text{ k}$$

$$T_3 = T_{y3}/\sin\theta = (525.6 \text{ k})/(\sin 43.6^\circ) = 762.2 \text{ k}$$

$$T_{x3} = T_3\cos\theta = (762.2 \text{ k})(\cos 43.6^\circ) = 551.9 \text{ k}$$

- From the above calculations, the following pattern may be observed:

$$R_2 = 157.3 \text{ k (for beam B)}$$

$$R_{21} = 213.04 \text{ k}$$

$$T_{yn} = R_{21} + (n-1)R_2 = 213.04 \text{ k} + (n-1)157.3 \text{ k}$$

$$T_n = T_{yn}/\sin\theta$$

$$T_{xn} = T_n\cos\theta$$

where n = floors 1 through 12

- It may now easily be determined what the forces will be at the top of the structure, or what loads the trusses must support:

- for beam A:

$$R_2 = 114.2 \text{ k}$$

$$T_{y12} = 213.04 \text{ k} + (12-1)114.2 \text{ k} = \underline{1469.24 \text{ k}}$$

$$T_{12} = (1469.24 \text{ k})/(\sin 43.6) = \underline{2130.5 \text{ k}}$$

$$T_{x12} = (2130.5 \text{ k})(\cos 43.6) = \underline{1542.9 \text{ k}}$$

- for beam B:

$$R_2 = 157.3 \text{ k}$$

$$T_{y12} = 213.04 \text{ k} + (12-1)157.3 \text{ k} = \underline{1932.34 \text{ k}}$$

$$T_{12} = (1932.34 \text{ k})/(\sin 43.6) = \underline{2802.0 \text{ k}}$$

$$T_{x12} = (2802 \text{ k})(\cos 43.6) = \underline{2029.2 \text{ k}}$$

13.5.2 Selection of Tension Members

There are several options to choose from for tension members: round bars or rods, plates, square or round pipes, and rolled or built-up sections. Typically, high-strength steel with tensile strengths on the order of 150 ksi are used for tension members. Generally, the design of tension members is quite straight forward; because the full cross-sectional area is effective in tension, reductions in allowable stresses are not required. For the investigation

here, the effect of the tension member's weight on bending will be neglected and pure tensile forces will be assumed.

The total axial tensile force acting on the extreme tension member at B at the top is:

$$(T_{12})_B = 2802.0 \text{ k}$$

- the required cross-sectional area is:

$$A_{\text{req.}} = T/F_t = (2802 \text{ k})/(0.6(150 \text{ ksi})) = \underline{31.13 \text{ in}^2}$$

- if it is desired to use a round bar, the required diameter will be:

$$\begin{aligned}\phi &= 2\sqrt{\frac{A}{\pi}} = 2\sqrt{\frac{31.13}{\pi}} \\ &= 6.3''\end{aligned}$$

- if it is desired to use a cable, the required area will be found based on 270K grade seven-wire strands with $F_t = 121.5 \text{ ksi}$ (refer to section 5.4.3), or:

$$A_{\text{req.}} = T/F_t = (2802 \text{ k})/(121.5 \text{ ksi}) = 23.06 \text{ in}^2$$

- the required diameter for a steel cable is:

$$\begin{aligned}\phi &= 2\sqrt{\frac{A}{\pi}} = 2\sqrt{\frac{23.06}{\pi}} \\ &= 5.42''\end{aligned}$$

For available rolled sections and referring to the AISC double-angles tables, a pair of L8x8x1.125 ($A = 33.5 \text{ in}^2$) may be used. These members are quite large and heavy. It should be realized that whichever type of solid section is chosen, it will have a weight of at least 106

pounds per lineal foot. The cable will have a weight of approximately 80 pounds per lineal foot. Thus, assuming the use of a cable system, the approximate load contribution of the tension members onto the top trusses will be:

$$\begin{aligned}\text{weight/member} &= (0.08 \text{ k/ft})(21.75') = 1.74 \text{ k/member} \\ T_y &= (1.74 \text{ k/member})(12 \text{ members}) \\ &= 20.88 \text{ k}\end{aligned}$$

Thus, the total load on the top trusses (truss B) will be:

$\begin{aligned}T_{yT} &= 1932.34 \text{ k} + 20.88 \text{ k} = \underline{1953.22 \text{ k}} \\ T_T &= (1953.22 \text{ k})/(\sin 43.6) = \underline{2832.32 \text{ k}} \\ T_{xT} &= (2832.32 \text{ k})(\cos 43.6) = \underline{2051.08 \text{ k}}\end{aligned}$

13.6 Moments Induced in Floor Levels by Differential Horizontal Force Components

It should be noted that at each level, the resultant of the horizontal forces, T_{xn} and $T_{x(n-1)}$, is not equal to zero. This is because the structural system as arranged is not capable of directly resisting the horizontal forces at the locations where they occur, but must resist them in bending along the length of the floor. This bending will induce moments at the core end of the floors. As was mentioned earlier, the floor system will be rigid enough to adequately resist the bending without warping such that the induced moments will be resisted by the pair of deep beams, A and B, in tension and compression.

The moments will be induced by the sum of the horizontal resultant forces from each of the beams A and B. For simplicity, it shall be generally assumed that beam A and B will contribute equally to the load and that the contribution shall be the value on beam B.

- at any floor, n:

$$(T_{xn})_T = 2T_{xn} - 2T_{x(n-1)}$$

- thus, from calculation:

$$\begin{aligned}(T_{x1})_T &= 2T_{x2} - 2T_{x1} \\ &= 2(388.85 \text{ k} - 223.7 \text{ k}) = \underline{330 \text{ k}}\end{aligned}$$

$$\begin{aligned}(T_{xn})_T &= 2T_{xn} - 2T_{x(n-1)} \\ &= \underline{330 \text{ k}}\end{aligned}$$

$$\begin{aligned}(T_{x13})_T &= 2T_{x13} - 2T_{x12} \\ &= 2(0 \text{ k}) - 2(2051 \text{ k}) = \underline{4102.2 \text{ k}}\end{aligned}$$

It can be seen that the moment which must be resisted by the top structure will be equal to the sum of the moments of the twelve floors beneath. The magnitude of the moments will be as follows:

$$M_n = ((T_{xn})_T)L = (55.9')((T_{xn})_T)$$

$$M_1 = (55.9')((T_{x1})_T) = (55.9')(330 \text{ k}) = \underline{18447 \text{ 'k}}$$

$$M_n = (55.9')(330 \text{ k}) = \underline{18447 \text{ 'k}} \text{ (n = 2 to 12)}$$

$$M_{13} = (55.9')(4102.2 \text{ k}) = \underline{229310.7 \text{ 'k}}$$

The trusses will also have to resist moment due to the vertical forces, T_{y13} , which shall be discussed shortly.

13.6.1 Axial Forces in Beams A and B Induced by Moments and Redetermination of Member Sizes

Beam A will exhibit combined compression and bending while beam B will exhibit combined tension and bending. The combined effects of compression and bending reduce the effectiveness of the section in resisting either action whereas tension and bending is not a critical combination. Thus, beam A is the critical member and shall be investigated first. Initially, the bending action due to the vertical loading will be neglected. The braced lengths will be assumed to be $L_y = 10$ feet in the weak direction and $L_x = 55.9$ feet in the strong direction. The effective length factor, K , of the end conditions will be assumed to be $K_x = 1.0$ (pin connections) in the strong direction and $K_y = 0.65$ (ideally fixed) in the weak direction.¹ Thus, the effective lengths are:

$$L'_x = K_x L_x = (1.0)(55.9') = 55.9'$$

$$L'_y = K_y L_y = (0.65)(10') = 6.5'$$

- *First floor (storage room)*

¹

Salmon, C., Johnson, J., Steel Structures: Design and Behavior, Second edition, Harper and Row, 1980, pp. 278.

W36x328:

$$I = 22500 \text{ in}^4, A = 96.4 \text{ in}^2, r_x = 15.3", r_y = 3.84", S = 1210 \text{ in}^3$$

$$M = 18447 \text{ 'k}$$

$$\rightarrow T/C = M/d = (18447 \text{ 'k})/20' = \underline{922.35 \text{ k}}$$

$$\rightarrow \rightarrow f_a = C/A = (922.35 \text{ k})/(96.4 \text{ in}^2) = 9.57 \text{ ksi}$$

- check which direction controls

$$r_x/r_y = 15.3"/3.84" = 3.98$$

- the equivalent effective length for the strong axis is:

$$L'_x = L'_y/(r_x/r_y) = 55.9'/3.98 = 13.95' > L'_y$$

→ strong axis (x-axis) controls¹

- determination of allowable axial stress, F_a

$$(KL/r)_x = 55.9'/15.3' = 3.63$$

→ from AISC table C-50, p. 3-17, $F_a = 29.75 \text{ ksi} > f_a \rightarrow \text{OK}$

- check combined bending and compression

$$M_b = 2912.56 \text{ 'k}$$

$$f_b = M_b/S = ((2912.56 \text{ 'k})/(1210 \text{ in}^3))(12"/') = 28.88 \text{ ksi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

¹

Salmon, C., Johnson, J., Steel Structures: Design and Behavior, Second edition, Harper and Row, 1980, pp. 282.

$$= \frac{9.57 \text{ ksi}}{29.75 \text{ ksi}} + \frac{28.88 \text{ ksi}}{30 \text{ ksi}} = 1.3 > 1$$

→ N.G. - must select new section

- try W36x439

$$I = 31000 \text{ in}^4, A = 128 \text{ in}^2, r_x = 15.6" , r_y = 3.95", S = 1620 \text{ in}^3$$

$$f_a = (922.35 \text{ k}) / (128 \text{ in}^2) = 7.21 \text{ ksi}$$

- check which direction controls

$$r_x / r_y = 15.6" / 3.95" = 3.95$$

- the equivalent effective length for the strong axis is:

$$L'_x = L'_y / (r_x / r_y) = 55.9' / 3.95 = 14.15' > L'_y$$

→ strong axis (x-axis) controls

- determination of allowable axial stress, F_a

$$(KL/r)_x = 55.9' / 15.6" = 3.58$$

→ from AISC table C-50, p. 3-17, $F_a = 29.77 \text{ ksi} > f_a \rightarrow \text{OK}$

- check combined bending and compression

$$f_b = M_y / S = ((2912.56 \text{ 'k}) / (1620 \text{ in}^3)) (12 \text{ '}) = 21.57 \text{ ksi}$$

$$\frac{7.21 \text{ ksi}}{29.77 \text{ ksi}} + \frac{21.57 \text{ ksi}}{30 \text{ ksi}} = 0.96 < 1 \rightarrow \text{OK}$$

- check combined tension and bending in beam B

$$T = 922.35 \text{ k}$$

$$f_t = T / A = (922.35 \text{ k}) / (128 \text{ in}^2) = 7.21 \text{ ksi} < F_t \rightarrow \text{OK}$$

$$\frac{f_t}{F_t} + \frac{f_b}{F_b} \leq 1$$

$$= \frac{7.21 \text{ ksi}}{30 \text{ ksi}} + \frac{21.57 \text{ ksi}}{30 \text{ ksi}} = 0.95 < 1 \rightarrow \text{OK}$$

→ select W36x439 for first floor (note that this section is not available in the U.S.)

- floors 2 through 12

W36x182:

$$I = 11300 \text{ in}^4, A = 53.6 \text{ in}^2, r_x = 14.5", r_y = 2.55", S = 623 \text{ in}^3$$

$$M = 18447 \text{ 'k}$$

$$\rightarrow T/C = M/d = (18447 \text{ 'k})/20' = \underline{922.35 \text{ k}}$$

$$\rightarrow \rightarrow f_a = C/A = (922.35 \text{ k})/(53.6 \text{ in}^2) = 17.21 \text{ ksi}$$

- check which direction controls

$$r_x/r_y = 14.5"/2.55" = 5.68$$

- the equivalent effective length for the strong axis is:

$$L''_x = L'_x/(r_x/r_y) = 55.9'/5.68 = 9.83' > L'_y$$

→ strong axis (x-axis) controls

- determination of allowable axial stress, F_a

$$(KL/r)_x = 55.9'/14.5' = 3.80$$

→ from AISC table C-50, p. 3-17, $F_a = 29.74 \text{ ksi} > f_a \rightarrow \text{OK}$

- check combined bending and compression

$$M_b = 1528.8 \text{ 'k}$$

$$f_b = M_b/S = ((1528.8 \text{ 'k})/(623 \text{ in}^3))(12"/') = 29.45 \text{ ksi}$$

$$\frac{\text{ksi}}{29.74 \text{ ksi}} + \frac{29.45 \text{ ksi}}{30 \text{ ksi}} = 1.56 > 1 \rightarrow N.G.$$

- try W36x300

$$I = 20300 \text{ in}^4, A = 88.3 \text{ in}^2, r_x = 15.2", r_y = 3.83", S = 1110 \text{ in}^3$$

$$f_a = C/A = (922.35 \text{ k})/(88.3 \text{ in}^2) = 10.44 \text{ ksi}$$

- check which direction controls

$$r_x/r_y = 15.2"/3.83" = 3.97$$

- the equivalent effective length for the strong axis is:

$$L''_x = L'_x/(r_x/r_y) = 55.9'/3.97 = 14.08' > L'_y$$

→ strong axis (x-axis) controls¹

- determination of allowable axial stress, F_a

$$(KL/r)_x = 55.9'/15.2' = 3.67$$

→ from AISC table C-50, p. 3-17, $F_a = 29.75 \text{ ksi} > f_a \rightarrow \text{OK}$

- check combined bending and compression

$$M_b = 1528.8 \text{ 'k}$$

¹

Salmon, C., Johnson, J., Steel Structures: Design and Behavior, Second edition, Harper and Row, 1980, pp. 282.

$$f_b = M_b/S = ((1528.8 \text{ 'k})/(1110 \text{ in}^3))(12"/') = 16.53 \text{ ksi}$$

$$\frac{\text{ksi}}{29.75 \text{ ksi}} + \frac{16.53 \text{ ksi}}{30 \text{ ksi}} = 0.9 < 1 \rightarrow OK$$

- check combined tension and bending

$$T = 922.35 \text{ k}$$

$$f_t = T/A = (793.8 \text{ k})/(88.3 \text{ in}^2) = 10.44 \text{ ksi} < F_t \rightarrow OK$$

$$\frac{10.44 \text{ ksi}}{30 \text{ ksi}} + \frac{16.53 \text{ ksi}}{30 \text{ ksi}} = 0.9 < 1 \rightarrow OK$$

→select W36x300 for units

13.7 Trusses

Only the B-truss, the critical truss, will be investigated here. As was mentioned earlier, each truss must resist the moment induced by the vertical component of the gravity loads, as well as its own weight. Furthermore, as a pair, they must resist the moments induced by the horizontal component of the gravity loads.

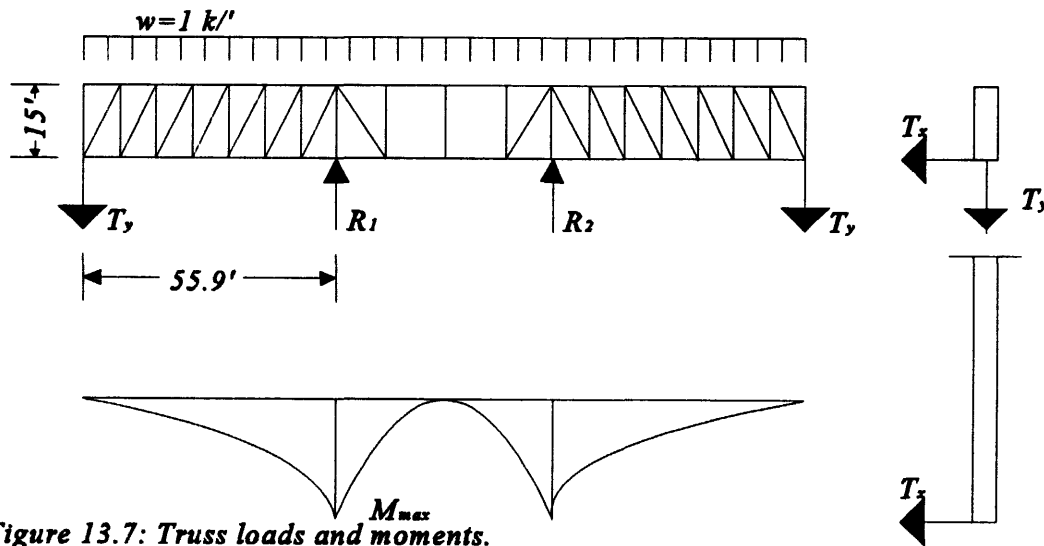


Figure 13.7: Truss loads and moments.

- assume self-weight of truss $w = 1 \text{ k/ft}$ acting on top chord

$$T_x = \underline{2051 \text{ k}}$$

$$T_y = \underline{1953.22 \text{ k}}$$

$$\begin{aligned} R_1 = R_2 &= T_y + 1/2wL = 1953.22 \text{ k} + 1/2(1 \text{ k/ft})(1778.8') \\ &= \underline{2843 \text{ k}} \end{aligned}$$

$$\begin{aligned} M_{\max} &= T_yL + wL^2/2 = (1953 \text{ k})(55.9') + (1 \text{ k/ft})(55.9')^2/2 \\ &= \underline{110747.4 \text{ 'k}} \end{aligned}$$

$$C/T_{\max} = M_{\max}/d = (110747.4 \text{ 'k})/(15') = \underline{7383 \text{ k}\pm}$$

13.7.1 Truss Member Forces

The method of sections will be used to determine the axial forces of the truss members at three critical locations. Refer to Appendix 1A for the calculation of the forces.

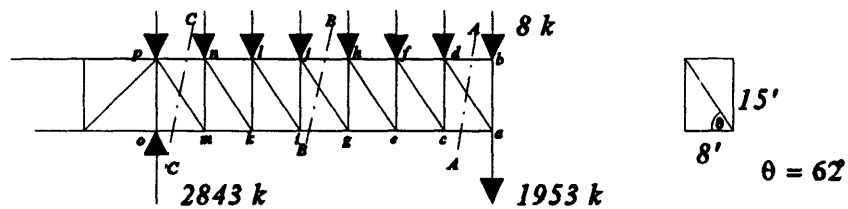


Figure 13.8: Truss member forces.

- section A-A

$$F_{ad} = 2253 \text{ k}$$

$$F_{bd} = 0 \text{ k}$$

$$F_{ac} = -1057.5 \text{ k}$$

- section B-B

$$F_{gj} = 2280 \text{ k}$$

$$F_{hj} = 3195.2 \text{ k}$$

$$F_{gi} = -4265.6 \text{ k}$$

- section C-C

$$F_{mp} = 2307 \text{ k}$$

$$F_{np} = 6428.8 \text{ k}$$

$$F_{mo} = -7512 \text{ k}$$

13.7.2 Determination of Truss Member Sizes

As was calculated above, the top chord of the truss will experience a maximum tension force at the supported end of 6428.8 k. Thus, the critical member is at the support.

- required area of top chord member

$$A_{\text{req.}} = T/F_t = (6428.8 \text{ k})/(30 \text{ ksi}) = 214.3 \text{ in}^2$$

→ select W36x798 ($A = 234 \text{ in}^2$, $S = 2980 \text{ in}^3$)

$$f_t = T/A = (6428.8 \text{ k})/(234 \text{ in}^2) = 27.8 \text{ ksi}$$

- bending due to its own weight

$$M = wL^2/10 = (0.798 \text{ k}')/(8')^2/10 = 5.1 \text{ 'k}$$

$$f_b = M/S = ((5.1 \text{ 'k})/(2980 \text{ in}^3))(12"/') = 0.0017 \text{ ksi} < F_b \rightarrow \text{OK}$$

- check combined bending and tension

$$\frac{f_t}{F_t} + \frac{f_b}{F_b} = \frac{27.8 \text{ ksi}}{30 \text{ ksi}} + \frac{0.0017 \text{ ksi}}{30 \text{ ksi}} = 0.92 < 1 \rightarrow \text{OK}$$

→ use W36x798 for top chord (note that this section is not available in the US.

The bottom chord will experience a compression force of 7512 k.

- effective length

$$L = 8'$$

$$K = 1.2 \text{ (assume fixed at one end and free translation at other)}$$

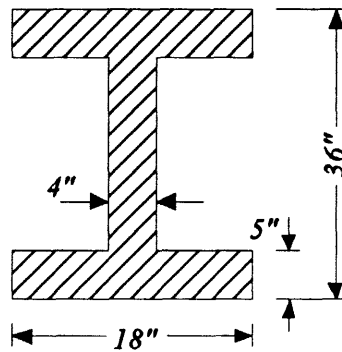
$$\rightarrow L' = KL = 1.2(8') = 9.6'$$

- minimum required area

$$A_{\text{req.}} = C/F_a = (7512 \text{ k})/(30 \text{ ksi}) = 250.4 \text{ in}^2$$

\rightarrow there is no rolled section with such an area

- try built-up section



$$A = 2(18'')(5'') + (4'')(36'' - 10'') \\ = 284 \text{ in}^2$$

$$r_x > 16.4'' \rightarrow (KL/r)_x < 1$$

$$\rightarrow F_a = 29.94 \text{ ksi}$$

$$S_x = (18''((36'')^3 - (26'')^3)/(6(36''))$$

$$+ (4''(26'')^2)/26''$$

$$= 2527.3 \text{ in}^3$$

$$w = (284 \text{ in}^2)(1'/12'')^2(1')(490 \text{ pcf}) = 0.966 \text{ k/ft}$$

$$f_a = C/A = (7607.8 \text{ k})/(284 \text{ in}^2) = 26.7 \text{ ksi} < F_a \rightarrow \text{OK}$$

$$M_b = wL^2/10 = (0.966 \text{ k/ft})(8')^2/10 = 6.18 \text{ 'k}$$

$$f_b = M/S = (6.18 \text{ 'k})/(2527.3 \text{ in}^3) = 0.002 \text{ ksi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{26.7 \text{ ksi}}{29.94 \text{ ksi}} + \frac{0.002 \text{ ksi}}{30 \text{ ksi}} = 0.89 < 1 \rightarrow \text{OK}$$

\rightarrow use above built-up section for bottom truss chord

All the diagonal members will be in tension, the critical value being 2307 k. Because the joints are pin-connected, bending will not be a factor.

$$A_{\text{req.}} = T/F_t = (2307 \text{ k})/(30 \text{ ksi}) = 76.9 \text{ in}^2$$

→ select and use W14x283 for diagonal members

The horizontal moments that the trusses must resist as a pair may be more effectively resisted by bracing the pair of trusses laterally against each other. The following figure illustrates only an example of one bracing scheme. The analysis will not be investigated in this paper.

13.8 Total Horizontal Force Components on Floors

For this portion of the investigation, it shall be assumed that the total horizontal force due to the combined action of each pair of trusses are equal.

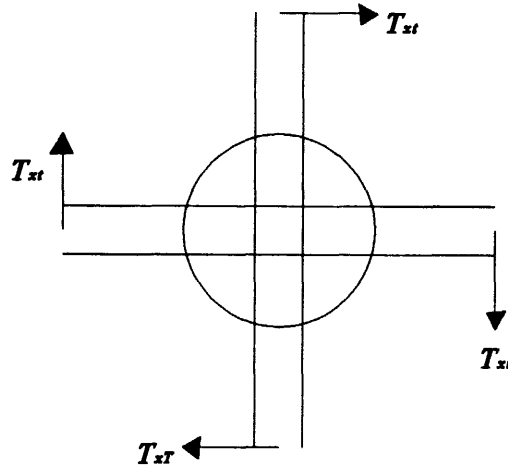


Figure 13.9: Torsional forces on floors.

- total horizontal force per pair of trusses

$$F_{xt13} = 4102 \text{ k}$$

$$F_{xt(2-12)} = 330 \text{ k}$$

$$F_{xt1} = 330 \text{ k}$$

- maximum total torsional moment at floor 13

$$T_{13} = 4F_{xt13}R = 4(4102 \text{ k})(35') = 558,120 \text{ 'k}$$

13.9 Determination of Core Thickness

13.9.1 Gravity Loads

$$U = 1.4D + 1.7L$$

It shall be assumed that the dead load is 75% of the total gravity loading.

- Point load at top transferred from trusses

$$P = R = 2075 \text{ k}$$

$$P_{DL} = 75\%(2075 \text{ k}) = 1556.25 \text{ k}$$

$$P_{LL} = 25\%(2075 \text{ k}) = 518.75 \text{ k}$$

$$P_U = 1.4(1556.25 \text{ k}) + 1.7(518.75 \text{ k}) = \underline{3060.6 \text{ k}}$$

- load is a bearing load on the concrete $\rightarrow \phi = 0.7$

$$\rightarrow f'_c = 0.7f_c = 0.7(8 \text{ ksi}) = 5.6 \text{ ksi}$$

$$\begin{aligned} A_{\text{req.}} &= P_U / f'_c = ((3060.6 \text{ k}) / (5.6 \text{ ksi})) (1'/12\text{in})^2 = 3.8 \text{ ft}^2 \\ &= 1.95' \times 1.95' \end{aligned}$$

$$\rightarrow \underline{t_{\text{min}} = 2'}$$

- Total building weight on core

$$W = 28931.5 \text{ k (refer to section 11.1.2)}$$

- weight of core

$$\begin{aligned} A &= \pi(r^2 - (r-t)^2) - 4((10')t) \\ &= \pi((35')^2 - (35'-2')^2) - 4(10')(2') = 347.26 \text{ ft}^2 \end{aligned}$$

$$V = Ah = (347.26 \text{ ft}^2)(235') = 81605.3 \text{ ft}^3$$

$$W_c = (0.150 \text{ kcf})V = (0.15 \text{ kcf})(81605.3 \text{ ft}^3) = \underline{12240.8 \text{ k}}$$

- total factored load

$$P_{DL} = 75\%(28931.5 \text{ k}) + 12240.8 \text{ k} = 33939.4 \text{ k}$$

$$P_{LL} = 25\%(28931.5 \text{ k}) = 7232.9 \text{ k}$$

$$P_U = 1.4(33939.4 \text{ k}) + 1.7(7232.9 \text{ k}) = \underline{59811.0 \text{ k}}$$

- load is a compression load on the concrete $\rightarrow \phi = 0.75$

$$\rightarrow f'_c = 0.75f_c = 0.75(8 \text{ ksi}) = 6 \text{ ksi}$$

$$A_{req.} = P_U / f'_c = ((59811 \text{ k}) / (6 \text{ ksi}))(12 \text{ in})^2 = 69.2 \text{ ft}^2 < A$$

$$\rightarrow \underline{t_{min} = 2'}$$

13.9.2 Torsional Loads

The total horizontal forces computed in section 13.8 will be transferred by floor diaphragm action to the core and will thus induce torsional forces in the core walls. From the bottom up, the torsional forces will increase, being maximum at the top, as calculated in section 13.8.

Torsional resistance of concrete is increased when the section experiences a compressive stress while under torsional load. The combined strength is calculated as follows (refer to section 10.2.2.2):

- combined shear and compression strength

$$f_v = 0.002(1 + 0.0005 \frac{P_U}{A}) \sqrt{f'_c}$$

$$= 0.002(1 + 0.0005 \frac{5981100 \text{ lb}}{(350 \text{ ft}^2)(12 \text{ in})^2}) \sqrt{8000}$$

$$= 0.29 \text{ ksi}$$

- load is shear and torsion $\rightarrow \phi = 0.85$

$$f'_v = 0.85f_v = 0.85(0.29 \text{ ksi}) = 0.25 \text{ ksi}$$

- minimum thickness required (refer to equation 4.16)

$$A_o = \pi r^2 = \pi (35')^2 = 3848.5 \text{ ft}^2$$

$$t_{\text{req.}} = T / (2A_o f'_v) = (568120 \text{ 'k}) / (2(3848.5 \text{ ft}^2)(0.25 \text{ ksi}))(1'/12'')^2 \\ = 2.0'$$

$$\rightarrow \underline{\text{minimum } t = 2'}$$

13.9.3 Lateral Loading (wind)

The wind load as calculated in section 11.2.1 is:

$$W = 6012.5 \text{ k}$$

- Bending

- total building weight

$$A = \pi(r^2 - (r-t)^2) - 4((10')t)$$

$$= \pi((35')^2 - (35'-2')^2) - 4(10')(2') = 347.26 \text{ ft}^2$$

$$V = Ah = (347.26 \text{ ft}^2)(235') = 81605.3 \text{ ft}^3$$

$$W_c = (0.150 \text{ kcf})V = (0.15 \text{ kcf})(81605.3 \text{ ft}^3) = 12240.8 \text{ k}$$

$$W_D = 12240.8 \text{ k} + 75\%(28931.5 \text{ k}) = 33939.4 \text{ k}$$

$$W_L = 25\%(28931.5 \text{ k}) = 7232.9 \text{ k}$$

$$P_U = 1.4(33939.4 \text{ k}) + 1.7(7232.9 \text{ k}) = 59811.12 \text{ k}$$

$$f_c = P_U / A = ((59811.12 \text{ k}) / (347.26 \text{ ft}^2))(1'/12'')^2 = 1.20 \text{ ksi}$$

- total factored load

$$H' = 1.3W = 1.3(6012.5 \text{ k}) = 7816.25 \text{ k}$$

$$M = H'h/2 = (7816.25 \text{ k})(235')/2 = \underline{918409.4 \text{ 'k}}$$

- section properties

$$c = 35'$$

$$I = \frac{\pi((70')^4 - (66')^4)}{64} - \frac{(10')((70')^3 - (66')^3)}{12} \\ - 2((2')(10')^3 + (10')(34')^2) \\ = 173794.6 \text{ ft}^4$$

- load is flexure $\rightarrow \phi = 0.9$

$$f'_c = 0.9f_c = 0.9(8 \text{ ksi}) = 7.2 \text{ ksi}$$

$$f_c = Mc/I = ((918409.4 \text{ 'k})(35')/(173794.6 \text{ ft}^4))('12")^2 \\ = 1.28 \text{ ksi}$$

$$f_{cT} = f_{cb} + f_{cc} = 1.28 \text{ ksi} + 1.20 \text{ ksi} = 2.48 \text{ ksi} < f'_c \rightarrow \text{OK}$$

- shear $\rightarrow \phi = 0.85 \rightarrow f'_c = 0.85f_c = 0.85(8 \text{ ksi}) = 6.8 \text{ ksi}$

$$f'_v = 0.25 \text{ ksi (from above)}$$

$$v = H'/A = ((7816.25 \text{ k})/(347.26 \text{ ft}^2))('12")^2 = 0.16 \text{ ksi}$$

$$v(\text{torsion}) = T/(2tA_o) = ((568120 \text{ 'k})/(2(2')(3848.5 \text{ ft}^2))('12")^2 \\ = 0.26 \text{ ksi}$$

$$v(\text{total}) = 0.16 \text{ ksi} + 0.26 \text{ ksi} = 0.42 \text{ ksi} > f'_v \rightarrow \text{N.G.}$$

- try thickness $t = 3'$

$$A = 511.5 \text{ ft}^2$$

$$v = H'/A = 0.11 \text{ ksi}$$

$$v(\text{torsion}) = T/(2tA_o) = 0.17 \text{ ksi}$$

$$v(\text{total}) = 0.11 + 0.17 = 0.28 \text{ ksi} < f'_v \rightarrow \text{OK}$$

- check lateral deflection

$$\Delta_{all} = h/500 = 235'(12''/')/500 = 5.64''$$

$$I = 259213.5 \text{ ft}^4$$

$$\Delta = \frac{wh^4}{8EI} = \frac{(25.6\text{k}') (235')^4}{8(5100\text{kst})(2592135\text{ft}^4)(12''/')} \\ = 5.64''$$

$$= 0.62" < \Delta_{all} \rightarrow \text{OK}$$

→ The approximate thickness of the core is 3'.

$$A = 511.5 \text{ ft}^2$$

$$V = (511.5 \text{ ft}^2)(235') = 120202.5 \text{ ft}^3 = 4452 \text{ CY}$$

13.10 Conclusions

To summarize the results of the investigation of scheme 1, the following member sizes were determined:

Beam A of units	W36x300
Beam B of units	W36x300
Tension members at top	24 in ² (80 lb/')
Top chord of truss	W36x798
Bottom chord of truss	Built-up section (966 lb/')
Diagonals of truss	W14x283
Core wall thickness	3'

Upon the preliminary investigation of scheme 1, it may be concluded that this alternative on its own will not be an efficient one. One problem is that many of the structural steel members determined for the various elements (the deep beams, truss chords, etc.) are not available in the United States as rolled sections or must be built-up. This would most definitely result in very high costs for the material alone. Another problem arises from the action of the structural system; the gravity loads alone induce large torsional moments in the concrete core. The required core thickness of three feet is a relatively large dimension for a structure of this use. The large thickness is the result of the combined effect of the torsional shear due to the gravity loads and the horizontal shear due to wind loads. Note that the torsional shear due to wind loads was not considered here, though this action will exist and will only add to the required core thickness. Furthermore, an analysis of the deflection and flexibility of the system was not performed. The flexibility of the cables, as well as their susceptibility to elongation, will cause difficulties in the design and construction of the structure.

Construction of the core may be difficult because of the relatively large thickness of the core wall. The construction may require much time and money, as well as a very sophisticated method of construction. One structure noted in section 8.1, the *National*

Westminster Bank Building, does have reinforced concrete walls that are nearly 4 feet thick and were slipformed, however, this is not the common practice.

A possible alternative to the 3' thick core walls could be to create a tube-in-tube type structure. In other words, a second reinforced concrete core may be placed within the main core. This inner core would house the vertical service shafts. The two cores would be made to act together by creating rigid diaphragms in the floor systems between the cores. This system would greatly increase the shear resistance on the structure as well as the torsional resistance. However, the inner core would not add to the flexural rigidity of the building, though this is not a concern here. A tube-in-tube system would allow for a reduction in the thickness of the main core walls, especially due to the fact that the thickness was determined by the shear stresses.

It may be concluded that scheme 1, as analyzed, as a structural system will require an unusually large amount of material (steel and concrete) and will require more complex construction methods.

Some numbers may be useful in making the comparisons of the different schemes. Considering the beams determined for the living units (W36x300) will apply for all floors, the total dead weight of the structure including dead loads from section 11.1.2 and structural material weights is:

Dead load	$= 75\%(28931.5 \text{ k}) = 21698.63 \text{ k}$
Weight per beam	$= (0.300 \text{ k}') (55.9' / \text{beam}) = 16.77 \text{ k}$
Weight per tension member	$= (0.08 \text{ k}') (21.75' / \text{tension member}) = 1.74 \text{ k}$
Weight per truss	$= (0.798 \text{ k}') (178.8' / \text{truss})$ $+ (0.966 \text{ k}') (178.8' / \text{truss})$ $+ (0.283 \text{ k}') (17' / \text{diagonal}) (14 \text{ diagonals} / \text{truss})$ $= 382.76 \text{ k}$
Weight of core	$= (0.150 \text{ kcf}) (511.5 \text{ ft}^2) (235') = 18030.4 \text{ k}$

$$\begin{aligned}
 \text{Total building weight} &= 21698.63 \text{ k} \\
 &+ (16.77 \text{ k/beam})(8 \text{ beams/floor})(12 \text{ floors}) \\
 &+ (1.74 \text{ k/tension})(8 \text{ tension/floor})(12 \text{ floors}) \\
 &+ (382.76 \text{ k/truss})(4 \text{ trusses}) \\
 &+ 18030.4 \text{ k} \\
 &= \underline{43043.7 \text{ k}}
 \end{aligned}$$

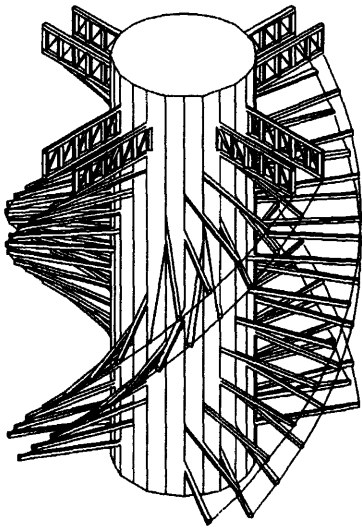


Figure 13.10: General view of suspension system structure.

CHAPTER 14

Scheme 2: Story-Height Vierendeel Trusses

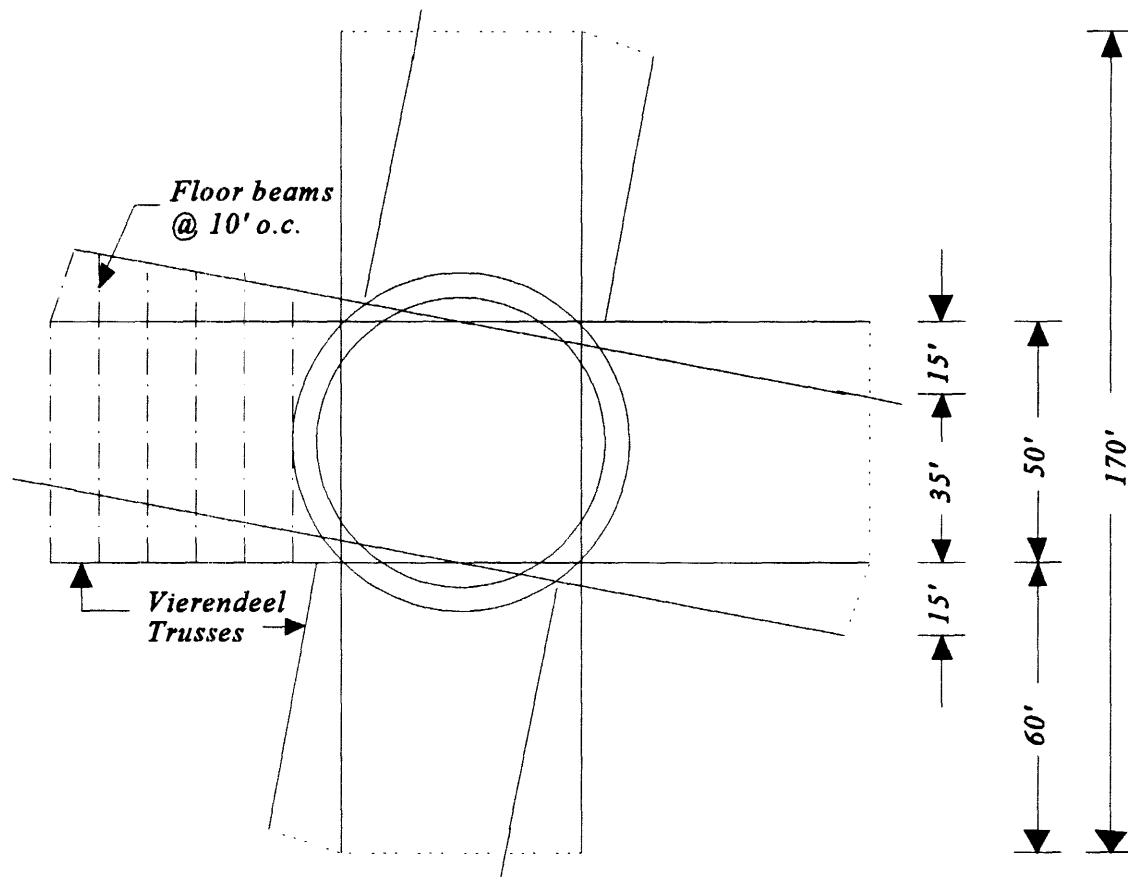


Figure 14.1: Plan dimensions of Vierendeel truss system

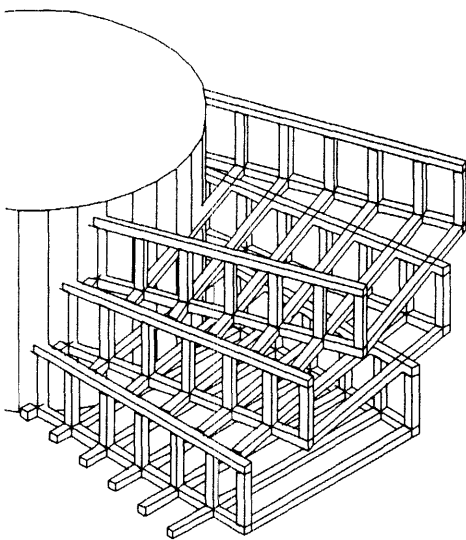


Figure 14.2: General view of vierendeel truss structural system

14.1 Determination of Tributary Areas

Only the floor level consisting of living units shall be considered here. Refer to figures 10.2 and 14.3 for determination of tributary areas.

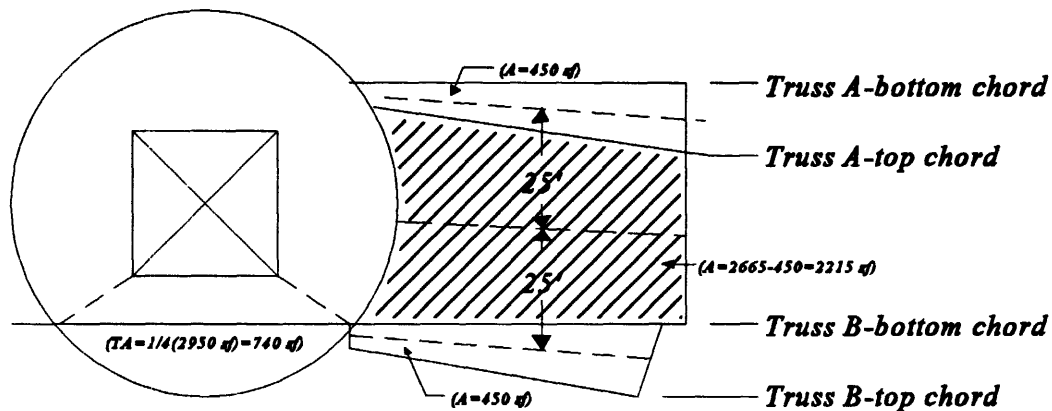


Figure 14.3: Tributary areas

- *Living Units*

- Truss A

$$TA_{top} = 1/2(450 \text{ sf}) + 1/2(2215 \text{ sf}) = 1333 \text{ sf}$$

$$TA_{bot} = 1/2(450 \text{ sf}) = 225 \text{ sf}$$

$$tw_{top} = 25' (@0.134 \text{ ksf})$$

$$tw_{bot} = 2.5' \text{ to } 7.5' \rightarrow 6' (@0.134 \text{ ksf})$$

- Truss B

$$TA_{top} = 1/2(450 \text{ sf}) = 225 \text{ sf}$$

$$TA_{bot} = 1/2(450 \text{ sf}) + 1/2(2215 \text{ sf}) = 1333 \text{ sf}$$

$$tw_{top} = 2.5' \text{ to } 7.5' \rightarrow 6' (@0.146 \text{ ksf})$$

$$tw_{\text{bot}} = 20' (@0.134 \text{ ksf}) + 6' (@ 0.146 \text{ ksf})$$

- *Lobbies*

$$TA = 740 \text{ sf}$$

$$tw = 0 \text{ to } 20' (@1/2(0.150 \text{ ksf}) = 0.075 \text{ ksf})$$

14.2 Gravity Loads on Trusses

Assume a dead load of 0.15 k' for the weight of the steel members.

- *Living Units*

- Truss A

- loads acting on top chord
 - distributed dead and live load of 0.134 ksf
 - concentrated dead load of 20 psf x 10' x 25' = 5 k from exterior wall
 - distributed dead load of 0.15 k'
 - loads acting on bottom chord
 - distributed dead and live load of 0.134 ksf
 - distributed dead load of 20 psf x 10' = 0.2 k' from exterior wall
 - concentrated dead load of 20 psf x 10' x 7.5' = 1.5 k
 - distributed dead load of 0.15 k'

- total loads on truss A

$$w_{top} = (0.134 \text{ ksf} \times 25') + 0.15 \text{ k}' = \underline{3.5 \text{ k}'}$$

$$P_{top} = \underline{5 \text{ k}}$$

$$w_{bot} = (0.134 \text{ ksf} \times 6') + 0.2 \text{ k}' + 0.15 \text{ k}' = \underline{1.15 \text{ k}'}$$

$$P_{bot} = \underline{1.5 \text{ k}}$$

- Truss B

- loads acting on top chord
 - distributed dead and live load of 0.146 ksf
 - distributed dead load of 0.15 k'

- loads acting on bottom chord
 - distributed dead and live load of 0.134 ksf
 - distributed dead and live load of 0.146 k'
 - distributed dead load of 20 psf x 10' = 0.2 k' from exterior wall
 - concentrated dead load of 20 psf x 10' x 17.5' = 3.5 k from exterior wall
 - distributed dead load of 0.15 k'

- total loads on truss B

$$w_{top} = (0.146 \text{ ksf} \times 6') + 0.15 \text{ k}' = \underline{1.03 \text{ k}'}$$

$$w_{bot} = (0.134 \text{ ksf} \times 20') + (0.146 \text{ ksf} \times 6') + 0.2 \text{ k}' + 0.15 \text{ k}' = \underline{3.91 \text{ k}'}$$

$$P_{bot} = \underline{3.5 \text{ k}}$$

- Lobbies

- distributed dead and live load of 0.075 ksf
- distributed dead load of $1/2(20 \text{ psf} \times 10' \times 30'/50') = 0.06 \text{ k}'$ from interior walls
- distributed dead load of 0.15 k'

- total loads in lobbies

$$w = (0.075 \text{ ksf} \times \{0 \text{ to } 20'\}) + 0.15 \text{ k}' + 0.06 \text{ k}' = \underline{0.21 \text{ k}' \text{ to } 1.71 \text{ k}'}$$

14.3 Reactions From Dead and Live Loads

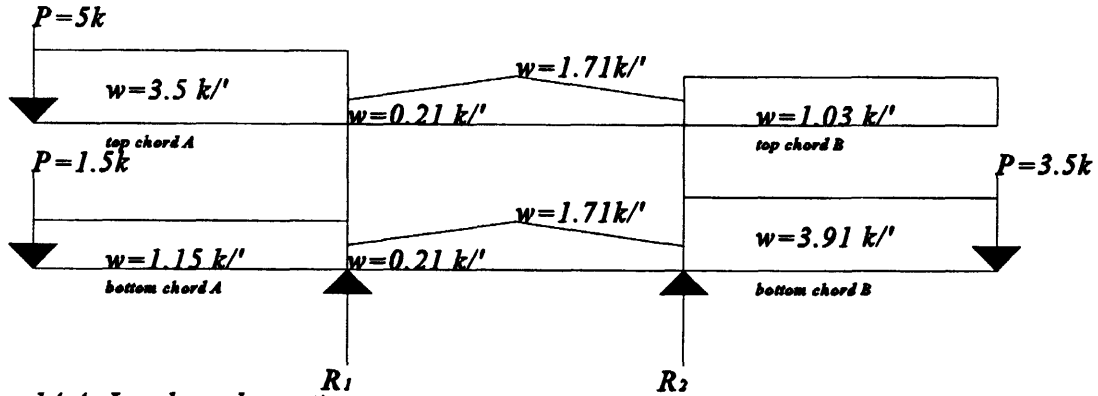


Figure 14.4: Loads and reactions on trusses

$$\begin{aligned}
 R_1 + R_2 &= 5k + 1.5k + 3.5k + (3.5k/ft + 1.15k/ft)(60') \\
 &\quad + (1.03k/ft + 3.91k/ft)(60') + 2((0.21k/ft)(50')) \\
 &\quad + 1/2(1.71k/ft - 0.21k/ft)(50')) \\
 &= 681.8 \text{ k}
 \end{aligned}$$

$$\begin{aligned}
 \Sigma M_{R1} &= (5k + 1.5k)(60') + 1/2(3.5k/ft + 1.15k/ft)(60')^2 \\
 &\quad - 2(1/2(0.21k/ft)(50')^2 + 1/2(1.71k/ft)(50')(50'/2)) \\
 &\quad - (3.5k)(110') - 1/2(3.91k/ft + 1.03k/ft)(60')^2 + R_2(50')
 \end{aligned}$$

$$\rightarrow R_2 = \underline{353.78 \text{ k}}$$

$$R_1 = \underline{328.02 \text{ k}}$$

$$\begin{aligned}
 M_{\max}(@R_2) &= wL^2/2 + PL \\
 &= (1.03 \text{ k/ft} + 3.91 \text{ k/ft})(60')^2/2 + (3.5 \text{ k})(60') \\
 &= \underline{9102 \text{ 'k}}
 \end{aligned}$$

This moment is resisted by the top and bottom chords in tension

and compression.

$$T/C = M/d = (9102 \text{ 'k})/(15') = 606.8 \text{ k}$$

This is a conservative estimate of the chord axial forces.

14.4 Truss Design

For the investigation of the trusses, only truss B shall be investigated because it is the most critically loaded. Because the floor beams are to be supported by the trusses at the joints, it will be assumed that all vertical loads act at the top and bottom joints and that these loads will be transferred to the top joints. The truss will be analyzed using the portal method for frame analysis. Refer to Appendix 2A for a more detailed explanation of the portal method of frame analysis. This method is commonly used for the preliminary design of frames and trusses.^{1, 2}

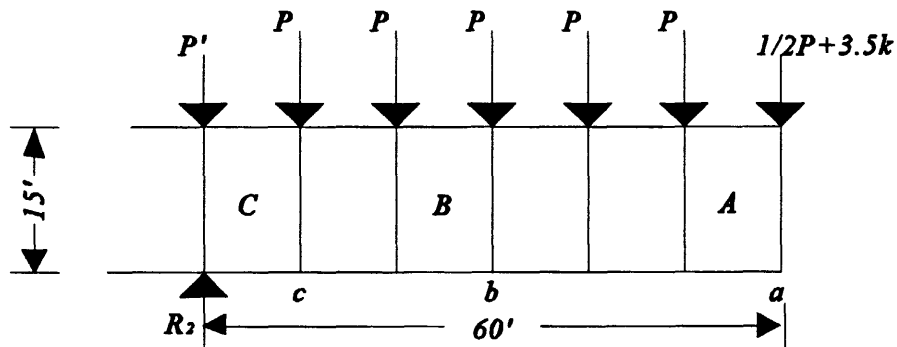


Figure 14.5: Loads on truss B

$$P = (w_{\text{top}} + w_{\text{bot}})L = (1.03 \text{ k}' + 3.91 \text{ k}') (60') = 49.4 \text{ k}$$

$$P' = 1/2(P + 3 \text{ k}) = 1/2(49.4 \text{ k} + 3 \text{ k}) = 26.2 \text{ k}$$

1

Schueller, W., High-rise Building Structures, Second edition, John Wiley and Sons, Inc., 1977.

2

Hsieh, Yuan-Yu, Elementary Theory of Structures, Third edition, Prentice Hall, 1988.

3

See Appendix 2B for calculation of lobby point loads.

14.4.1 Shear and Axial Forces in Chords

The following forces will act on the elements noted above (A, B, C). Refer to Appendix 2A for the calculation of these forces.

panel A

$$V = 14.1 \text{ k}$$

N = 9.4 k

- **panel B**

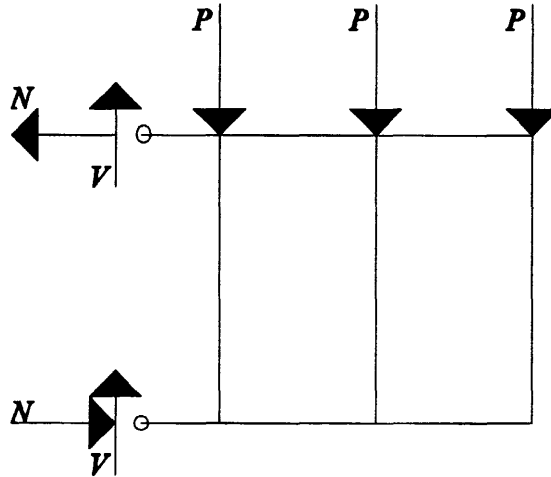
$$V = 88.2 \text{ k}$$

N = 214 k

- panel C

$$V = 137.6 \text{ k}$$

N = 515.1 k



14.4.2 Shear and Axial Forces in Columns and Moments in Chords and Columns

The following forces will act on the elements noted above (a, b, c). Refer to Appendix 2A for the calculation of these forces.

- **column a**

V = 9.4 k

N = 14.1 k

$$M_1 = 70.5 \text{ 'k}$$

$$M_2 = 70.5 \text{ 'k}$$

- column b

$$V = 101.1 \text{ k}$$

$$N = 24.7 \text{ k}$$

$$M_1 = 441 \text{ 'k}$$

$$M_2 = 758.5 \text{ 'k}$$

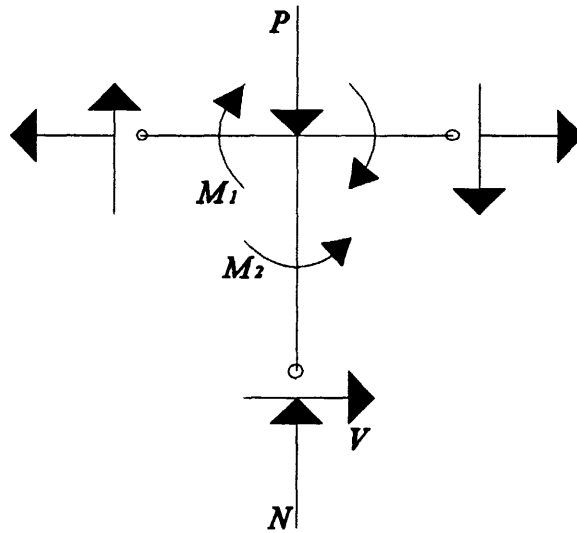
- column c

$$V = 167 \text{ k}$$

$$N = 24.7 \text{ k}$$

$$M_1 = 688 \text{ 'k}$$

$$M_2 = 1252.5 \text{ 'k}$$



- at support

$$V = 74.2 \text{ k}$$

$$N = 190 \text{ k}$$

$$M_1 = -131 \text{ 'k}$$

$$M_2 = 556.5 \text{ 'k}$$

14.4.3 Chord Member Sizing

Because the chords are fully laterally braced by the floor diaphragm, the allowable stresses may be taken as $F_a = 0.6F_y = 30 \text{ ksi}$ and $F_b = 0.66F_y = 33 \text{ ksi}$. For members that behave as beams and columns, or in bending and axial loading, as do the chord members, the following formula may be used to determine the required cross-sectional area:¹

$$A_{req.} = \frac{F + B_x M}{F_a}$$

¹

Schueller, W, The Vertical Building Structure, Van Nostrand Reinhold, 1992, pp. 286.

where B_x is the bending factor = $2.5/t$ for W-sections, and t is the depth of the section.

- panel A

$$F = 9.4 \text{ k}$$

$$M = 70.5 \text{ 'k}$$

try W10 section

$$A = ((9.4 \text{ k}) + (2.5/10'')(70.5 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 7.36 \text{ in}^2$$

$\rightarrow \text{use W10x26} - A = 7.61 \text{ in}^2, S = 27.9 \text{ in}^3$

To be sure that the member can resist the combined action of compression and bending, a check may be performed using the interaction equation (equation 5.3).

$$f_a = (9.4 \text{ k})/(7.61 \text{ in}^2) = 1.24 \text{ ksi}$$

$$f_b = (70.5 \text{ 'k})(12''/)/(27.9 \text{ ksi}) = 30.32 \text{ ksi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{1.24 \text{ ksi}}{30 \text{ ksi}} + \frac{30.32 \text{ ksi}}{33 \text{ ksi}} = 0.96 < 1 \rightarrow OK$$

For the calculation of the combined effects for the other members, please refer to Appendix 2C.

- panel B

$$F = 214 \text{ k}$$

$$M = 441 \text{ 'k}$$

try W14 section

$$A = ((214 \text{ k}) + (2.5/14'')(441 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 38.66 \text{ in}^2$$

→use W14x145 - $A = 42.7 \text{ in}^2$, $S = 323 \text{ in}^3$

- panel C

$$F = 515.5 \text{ k}$$

$$M = 688 \text{ 'k}$$

try W24 section

$$A = ((515.5 \text{ k}) + (2.5/24'')(688 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 45.9 \text{ in}^2$$

→use W24x162 - $A = 47.7 \text{ in}^2$, $S = 414 \text{ in}^3$

14.4.4 Column Member Sizing

The same principles as those above apply for the column members as well.

- column a

$$F = 14.1 \text{ k}$$

$$M = 70.5 \text{ 'k}$$

try W10 section

$$A = ((14.1 \text{ k}) + (2.5/10'')(70.5 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 7.52 \text{ in}^2$$

→use W10x26 - $A = 7.61 \text{ in}^2$, $S = 27.9 \text{ in}^3$

- column b

$$F = 24.7 \text{ k}$$

$$M = 758.5 \text{ 'k}$$

try W18 section

$$A = ((24.7 \text{ k}) + (2.5/18")(758.5 \text{ 'k})(12"/'))/(30 \text{ ksi}) = 42.96 \text{ in}^2$$

$$\rightarrow \text{use W18x158} - A = 46.3 \text{ in}^2, S = 310 \text{ in}^3$$

- column c

$$F = 24.7 \text{ k}$$

$$M = 1252.5 \text{ 'k}$$

In this case, the moment will clearly control the design, thus, the required section modulus will be determined with an increase of 10% to account for the axial load.

$$S = (1.1)M/F_b = (1.1)((1252.5 \text{ 'k})/(33 \text{ ksi}))(12"/') = 501 \text{ in}^3$$

$$\rightarrow \text{use W36x160} - A = 47 \text{ in}^2, S = 542 \text{ in}^3$$

- at support

$$F = 190 \text{ k}$$

$$M = 556.5 \text{ 'k}$$

try W24 section

$$A = ((190 \text{ k}) + (2.5/24"))(556.5 \text{ 'k})(12"/')/(30 \text{ ksi}) = 29.5 \text{ in}^2$$

→use W24x104 - $A = 30.6 \text{ in}^2$, $S = 258 \text{ in}^3$

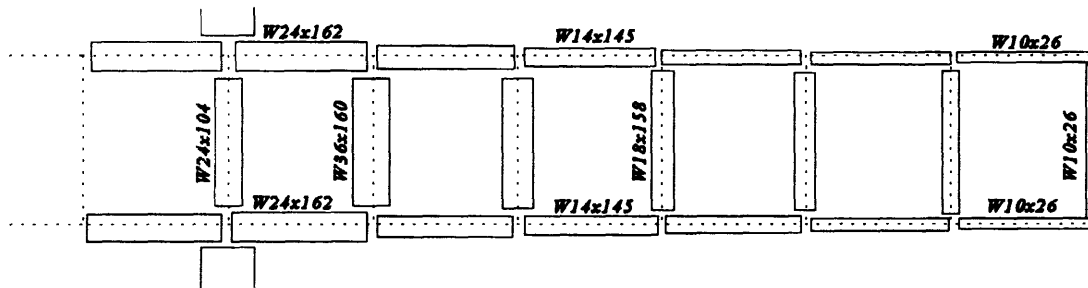


Figure 14.6: General truss member sizes

14.4.5 Truss Deflection Check

The deflection of the truss under gravity loading should be checked to assure that it is within the allowable limits. The maximum deflection allowed under dead and live loading on floor members is given by:

$$\Delta_{all} = L/240 = (60')(12"/')/240 = 3"$$

The deflection of a truss may be estimated in the same way as the lateral deflection of a rigid frame. The equation is as follows:¹

$$\Delta \sim \frac{2f_s H^2}{3EB}$$

where f_s is the maximum stress in the member at the support, H is the height or length, and

¹

Schueller, W., The Vertical Building Structure, Van Nostrand Reinhold, 1990, pp.497-500.

B is the distance between the supporting members. Thus:

$$f_a = N/A = (515.1 \text{ k})/(47.7 \text{ in}^2) = 10.80 \text{ ksi}$$

$$B = 15'$$

$$H = 60'$$

$$\begin{aligned}\Delta &= (2(10.8 \text{ ksi})(60')^2)/(3(29000 \text{ ksi})(15'))(12"/\text{ft}) \\ &= 0.72" < \Delta_{all} \rightarrow \text{OK}\end{aligned}$$

It should be noted that, due to the additional stiffness resulting from the adjacent floors being joined together, the deflection will be less than that just calculated.

14.5 Determination of Core Thickness

14.5.1 Gravity Loads

$$U = 1.4D + 1.7L$$

The dead load shall be assumed to be 75% of the total load.

- Point load transferred from each truss

$$P = R = 353.78 \text{ k}$$

$$P_D = 75\%P = 287.84 \text{ k}$$

$$P_L = 25\%P = 65.94 \text{ k}$$

$$P_U = 1.4P_D + 1.7P_L = 515.08 \text{ k}$$

- load is bearing on concrete $\rightarrow \phi = 0.7$

$$\rightarrow f'_c = 0.7f_c = 0.7(8 \text{ ksi}) = 5.6 \text{ ksi}$$

$$\begin{aligned} A_{\text{req.}} &= P_U / f'_c = (515.08 \text{ k}) / (5.6 \text{ ksi}) = 91.98 \text{ in}^2 \\ &= 9.6" \times 9.6" \end{aligned}$$

$$\rightarrow \underline{t_{\text{min}} = 12" = 1'}$$

- Total building weight on core

$$W = 28931.5 \text{ k}$$

- weight of core

$$\begin{aligned} A &= \pi(r^2 - (r-t)^2) - 4((10')t) \\ &= \pi((35')^2 - (35'-1')^2) - 4(10')(1') = 176.77 \text{ ft}^2 \end{aligned}$$

$$V = Ah = (176.77 \text{ ft}^2)(235') = 41540.92 \text{ ft}^3$$

$$W_c = (0.15 \text{ kcf})V = (0.15 \text{ kcf})(41540.92 \text{ ft}^3) = \underline{6231.14 \text{ k}}$$

- total factored load

$$P_D = 75\%W + W_c = 75\%(28931.5 \text{ k}) + 6231.14 \text{ k} = 27929.76 \text{ k}$$

$$P_L = 25\%W = 25\%(28931.5 \text{ k}) = 7232.9 \text{ k}$$

$$P_U = 1.4(27929.76 \text{ k}) + 1.7(7232.9 \text{ k}) = \underline{51397.60 \text{ k}}$$

- load is compression $\rightarrow \phi = 0.75$

$$\rightarrow f'_c = 0.75f_c = 0.75(8 \text{ ksi}) = 6 \text{ ksi}$$

$$A_{\text{req.}} = P_U / f'_c = ((51397.6 \text{ k}) / (6 \text{ ksi})) (/12")^2 = 57.8 \text{ ft}^2 < A$$

$$\rightarrow \underline{t_{\text{min}} = 12" = 1'}$$

14.5.2 Lateral loads

$$W = 6012.5 \text{ k}$$

- Bending

- total building weight

$$P_U = 51397.6 \text{ k}$$

$$f_c = P_U / A = ((51397.6 \text{ k}) / (176.77 \text{ ft}^2)) (/12")^2 = 2.02 \text{ ksi} < f'_c$$

- factored wind load

$$H = 1.3W = 1.3(6012.5 \text{ k}) = 7816.25 \text{ k}$$

$$M = Hh/2 = (7816.25 \text{ k})(235')/2 = 918409.4 \text{ 'k}$$

- section properties

$$c = 35'$$

$$\begin{aligned} I &= \frac{\pi((70')^4 - (68')^4)}{64} - \frac{(10')((70')^3 - (68')^3)}{12} \\ &\quad - 2((1')(10')^3 + (10')(34.5')^2) \\ &= 79420.6 \text{ ft}^4 \end{aligned}$$

- load is flexure $\rightarrow \phi = 0.9$

$$\rightarrow f'_c = 0.9f_c = 0.9(8 \text{ ksi}) = 7.2 \text{ ksi}$$

$$f_c = Mc/I = (918409.4 \text{ 'k})(35')/(79420.6 \text{ ft}^4)('/12'')^2 = 2.81 \text{ ksi}$$

$$f_{ct} = 2.02 \text{ ksi} + 2.81 \text{ ksi} = 4.83 \text{ ksi} < f'_c \rightarrow \text{OK}$$

- shear $\rightarrow \phi = 0.85$

$$\rightarrow f'_c = 0.85f_c = 0.85(8 \text{ ksi}) = 6.8 \text{ ksi}$$

$$f_v = 0.002(1 + 0.0005 \frac{51397600b}{(176.77 \text{ ft}^2)(12''/')^2}) \sqrt{8000 \text{ ksi}} = 0.36 \text{ ksi}$$

$$v = H/A = ((7816.25 \text{ k})/(176.77 \text{ ft}^2))(/12'')^2 = 0.31 < f'_v \rightarrow \text{OK}$$

- check lateral deflection

$$\Delta_{all} = h/500 = 5.64''$$

$$\Delta = \frac{wh^4}{8EI} = \frac{(25.6 \text{ k/'})(235')^4}{8(5100 \text{ ksi})(794206 \text{ ft}^4)(12''/')} \\ = 2.00'' < \Delta_{all} \rightarrow \text{OK}$$

\rightarrow The approximate thickness of the core will be 12"

$$A = 177 \text{ ft}^2$$

$$V = (177 \text{ ft}^2)(235') = 41541 \text{ ft}^3 = 1540 \text{ CY}$$

14.6 Conclusions

To summarize the results of the investigation of scheme 2, the following structural member sizes were determined:

Chord member A	W10x26
Chord member B	W14x145
Chord member C	W24x162
Column member a	W10x26
Column member b	W18x158
Column member c	W36x160
Column at support	W24x104
Core wall thickness	12"

The above values may be used to determine the total dead weight of the building:

Dead load	$= 75\%(28931.5 \text{ k}) = 21698.63 \text{ k}$
Weight per truss	$= (0.026 \text{ k}') (10' / \text{member}) (4 \text{ members / truss})$ $+ (\sim 0.075 \text{ k}') (10' / \text{member}) (8 \text{ members / truss})$ $+ (0.145 \text{ k}') (10' / \text{member}) (4 \text{ members / truss})$ $+ (\sim 0.150 \text{ k}') (10' / \text{member}) (4 \text{ members / truss})$ $+ (0.162 \text{ k}') (10' / \text{member}) (4 \text{ member / truss})$ $+ (0.026 \text{ k}') (15' / \text{member}) (2 \text{ member / truss})$ $+ (\sim 0.100 \text{ k}') (15' / \text{member}) (4 \text{ member / truss})$ $+ (0.158 \text{ k}') (15' / \text{member}) (2 \text{ member / truss})$ $+ (0.160 \text{ k}') (15' / \text{member}) (4 \text{ member / truss})$ $+ (0.104 \text{ k}') (15' / \text{member}) (2 \text{ member / truss})$ $+ (\sim 0.32 \text{ k}') (50')$

	$= 65.56 \text{ k}$
Weight of core	$= (0.150 \text{ kcf})(41540.92 \text{ ft}^3) = 6231.14 \text{ k}$
Total building weight	$= \underline{31338.88 \text{ k}}$

It may be seen that this building weighs quite a bit less than the scheme 1 building. Even though scheme 2 uses much more steel than scheme 1, the difference in building weight lies in the amount of concrete each one uses. Though torsion will be generated under wind loading, it is not generated under pure gravity loading, thus the smaller amount of required concrete.

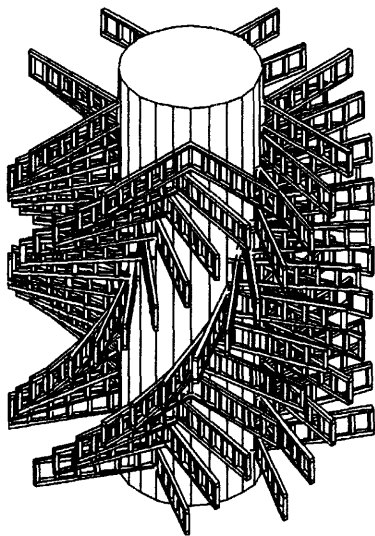


Figure 14.7: General view of vierendeel truss system

CHAPTER 15

Scheme 3: Story-Height Vierendeel Trusses on Alternating Floors

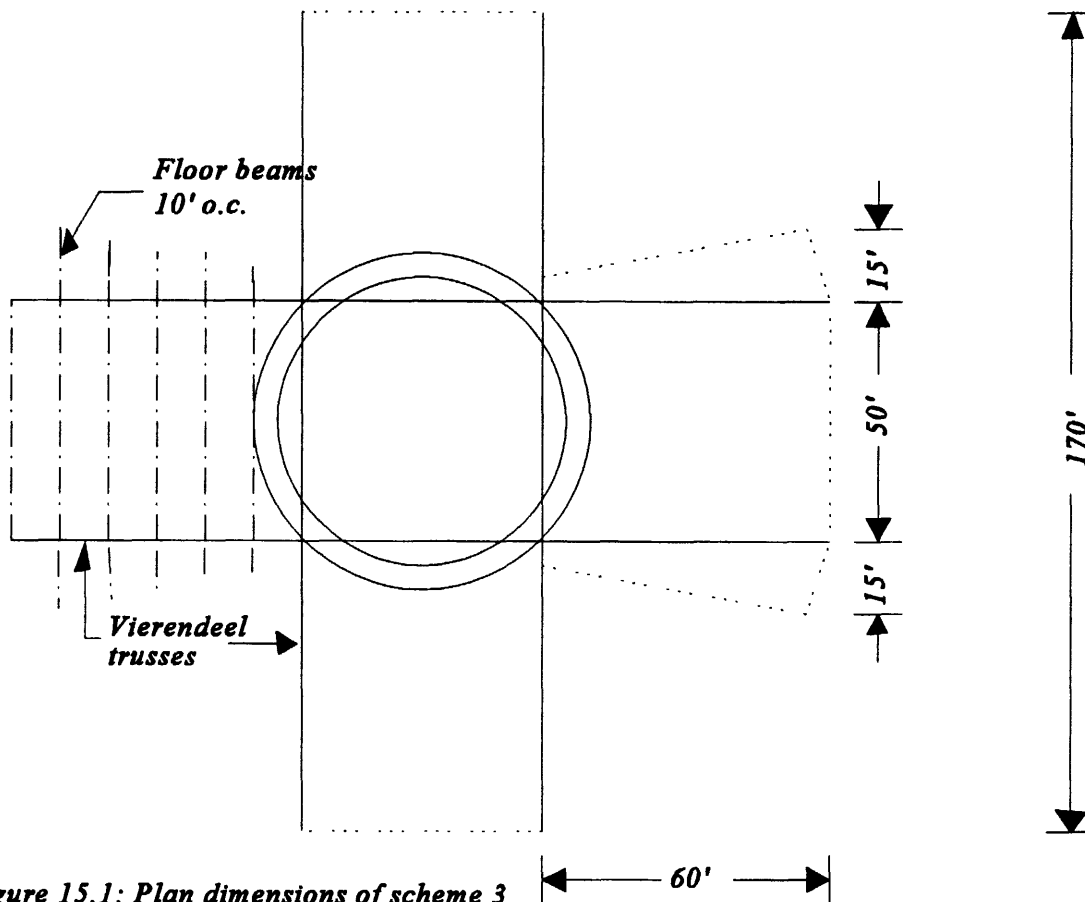


Figure 15.1: Plan dimensions of scheme 3

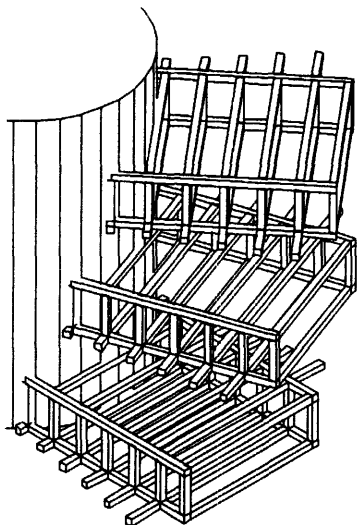


Figure 15.2: General view of structural system

15.1 Determination of Tributary Areas

Only the floor level consisting of living units shall be considered here.

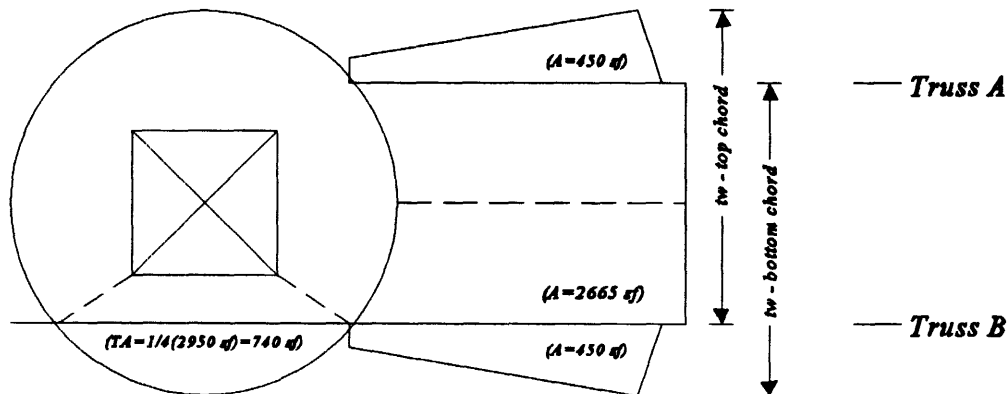


Figure 15.3: Tributary areas

- *Living Units*

- Truss A

$$TA_{\text{top}} = 450 \text{ sf} + 1/2(2665 \text{ sf}) = 1782.5 \text{ sf}$$

$$TA_{\text{bot}} = 1/2(2665 \text{ sf}) = 1332.5 \text{ sf}$$

$$tw_{\text{top}} = 30' \text{ to } 40' \rightarrow 37' (@0.134 \text{ ksf})$$

$$tw_{\text{bot}} = 25' (@0.134 \text{ ksf})$$

- Truss B

$$TA_{\text{top}} = 1/2(2665 \text{ sf}) = 1332.5 \text{ sf}$$

$$TA_{\text{bot}} = 450 \text{ sf} + 1/2(2665 \text{ sf}) = 1782.5 \text{ sf}$$

$$tw_{\text{top}} = 25' (@ 0.134 \text{ ksf and } 0.146 \text{ ksf})$$

$$tw_{\text{bot}} = 30' \text{ to } 40' \text{ } 20' \rightarrow 37' (@0.134 \text{ ksf and } 0.146 \text{ ksf})$$

- *Lobbies*

TA = 740 sf

tw = 0 to 20' (@ 0.150 ksf)

15.2 Gravity Loads on Trusses

Assume a dead load of 0.25 k' for the weight of the steel members.

- *Living Units*

- Truss A

- loads acting on top chord
 - distributed dead and live load of 0.134 ksf
 - distributed dead load of 20 psf x 10' = 0.2 k' from exterior wall
 - concentrated dead load of 20 psf x 10' x 25' = 5 k from exterior wall
 - distributed dead load of 0.25 k'
 - loads acting on bottom chord
 - distributed dead and live load of 0.134 ksf
 - distributed dead load of 20 psf x 10' = 0.2 k' from exterior wall
 - concentrated dead load of 20 psf x 10' x 25' = 5 k
 - distributed dead load of 0.25 k'

- total loads on truss A

$$w_{\text{top}} = (0.134 \text{ ksf} \times 37') + 0.2 \text{ k}' + 0.25 \text{ k}' = \underline{5.41 \text{ k}'}$$

$$P_{\text{top}} = \underline{5 \text{ k}}$$

$$w_{\text{bot}} = (0.134 \text{ ksf} \times 25') + 0.2 \text{ k}' + 0.25 \text{ k}' = \underline{3.8 \text{ k}'}$$

$$P_{\text{bot}} = \underline{5 \text{ k}}$$

- Truss B

- loads acting on top chord
 - distributed dead and live load of 0.146 ksf
 - distributed dead and live load of 0.134 ksf

- distributed dead load of $20 \text{ psf} \times 10' = 0.2 \text{ k'}$ from exterior wall
- concentrated dead load of $20 \text{ psf} \times 10' \times 25' = 5 \text{ k}$ from exterior wall
- distributed dead load of 0.25 k'
- loads acting on bottom chord
 - distributed dead and live load of 0.134 ksf
 - distributed dead and live load of 0.146 k'
 - distributed dead load of $20 \text{ psf} \times 10' = 0.2 \text{ k'}$ from exterior wall
 - concentrated dead load of $20 \text{ psf} \times 10' \times 25' = 5 \text{ k}$ from exterior wall
 - distributed dead load of 0.25 k'

- total loads on truss B

$$w_{\text{top}} = (0.134 \text{ ksf})(1/2(25')) + (0.146 \text{ ksf})(1/2(25')) + .02 \text{ k'}$$

$$+ 0.25 \text{ k'} = \underline{3.95 \text{ k'}}$$

$$P_{\text{top}} = 5 \text{ k}$$

$$w_{\text{bot}} = (0.134 \text{ ksf} \times 25') + (0.146 \text{ ksf} \times 12') + 0.2 \text{ k'} + 0.25 \text{ k'}$$

$$= \underline{5.55 \text{ k'}}$$

$$P_{\text{bot}} = \underline{5 \text{ k}}$$

- Lobbies

- distributed dead and live load of 0.15 ksf
- distributed dead load of $20 \text{ psf} \times 10' \times 30'/50' = 0.12 \text{ k'}$ from interior walls
- distributed dead load of 0.25 k'

- total loads in lobbies

$$w = (0.15 \text{ ksf} \times \{0 \text{ to } 20'\}) + 0.25 \text{ k'} + 0.12 \text{ k'} = \underline{0.37 \text{ to } 3.37 \text{ k'}}$$

15.3 Reactions From Dead and Live Loads

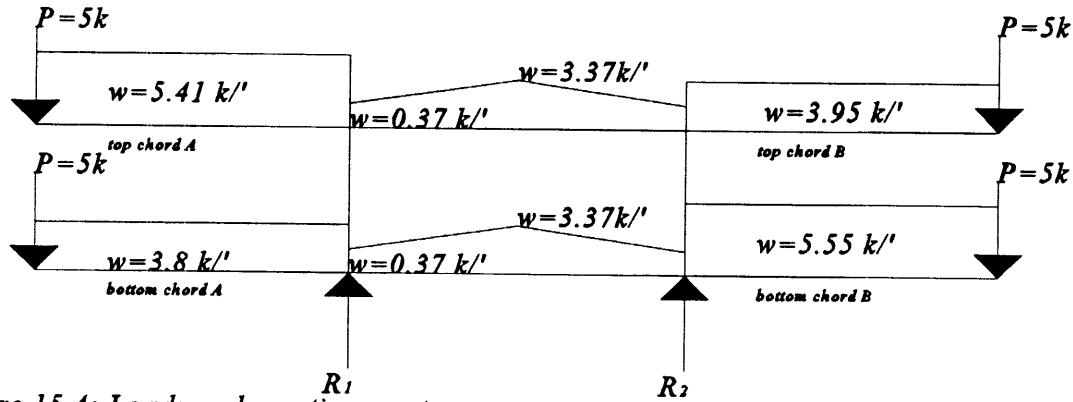


Figure 15.4: Loads and reactions on trusses

$$\begin{aligned}
 R_1 + R_2 &= 4(5 \text{ k}) + (5.41 \text{ k/ft} + 3.8 \text{ k/ft})(60') \\
 &\quad + (3.95 \text{ k/ft} + 5.55 \text{ k/ft})(60') + 2((0.37 \text{ k/ft})(50')) \\
 &\quad + 1/2(3.37 \text{ k/ft} - 0.37 \text{ k/ft})(50')) \\
 &= 1329.6 \text{ k}
 \end{aligned}$$

$$\begin{aligned}
 \Sigma M_{R1} &= 2(5 \text{ k})(60') + 1/2(5.41 \text{ k/ft} + 3.8 \text{ k/ft})(60')^2 \\
 &\quad - 2(5 \text{ k})(110') - (3.95 \text{ k/ft} + 5.55 \text{ k/ft})(60')(80') \\
 &\quad - 1/2(0.37 \text{ k/ft})(50')^2 - 2(1/2(50')(3.37 \text{ k/ft} - 0.37 \text{ k/ft}))(25') \\
 &\quad + R_2(50')
 \end{aligned}$$

$$\rightarrow R_2 = \underline{674.69 \text{ k}}$$

$$R_1 = \underline{654.91 \text{ k}}$$

$$\begin{aligned}
 M_{\max}(@R_2) &= wL^2/2 + PL \\
 &= (3.95 \text{ k/ft} + 5.55 \text{ k/ft})(60')^2/2 + 2(5 \text{ k})(60') \\
 &= \underline{17700 \text{ 'k}}
 \end{aligned}$$

This moment is resisted by the top and bottom chords in tension and compression.

$$T/C = M/d = (17700 \text{ 'k})/(15') = 1180 \text{ k}$$

This is a conservative estimate of the chord axial forces.

15.4 Truss Design

For the investigation of the trusses, only truss B shall be investigated because it is the most critically loaded. Because the floor beams are to be supported by the trusses at the joints, it will be assumed that all vertical loads act at the top and bottom joints and that these loads will be transferred to the top joints.

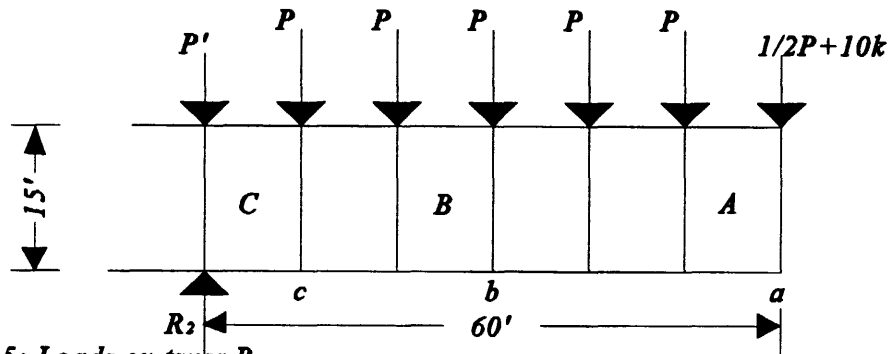


Figure 15.5: Loads on truss B

$$P = (w_{\text{top}} + w_{\text{bot}})L = (3.95 \text{ k}' + 5.55 \text{ k}')(60') = 95 \text{ k}$$

$$P' = 1/2P + 2(4.9 \text{ k}') = 1/2(95 \text{ k}) + 2(4.9 \text{ k}') = 57.3 \text{ k}$$

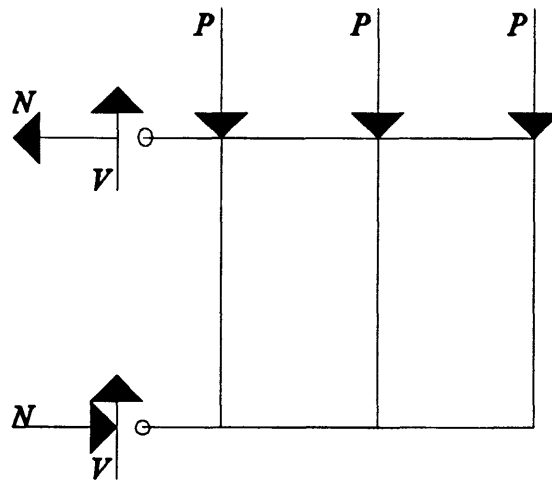
15.4.1 Shear and Axial Forces in Chords

The following forces will act on the elements noted above (A, B, C). Please refer to Appendix 3A for the calculation of these forces.

- panel A
 $V = 28.75 \text{ k}$
 $N = 19.2 \text{ k}$

- panel B
 $V = 171.25 \text{ k}$
 $N = 419.17 \text{ k}$

- panel C
 $V = 266.25 \text{ k}$
 $N = 1002.5 \text{ k}$

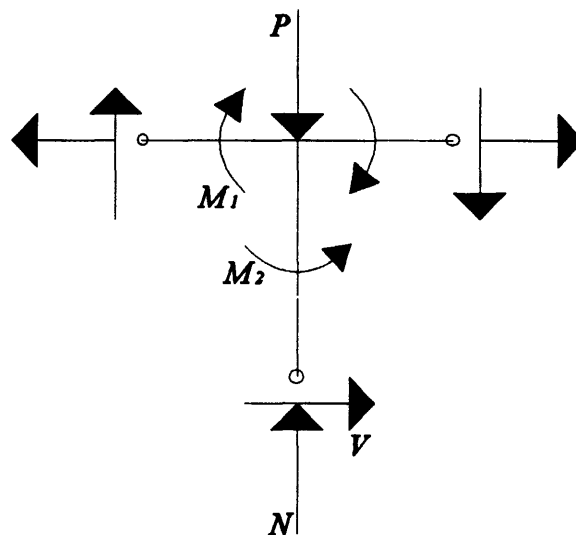


15.4.2 Shear and Axial Forces in Columns and Moments in Chords and Columns

The following forces will act on the elements noted above (a, b, c). Please refer to Appendix 3A for the calculation of these forces.

- column a
 $V = 19.2 \text{ k}$
 $N = 28.75 \text{ k}$
 $M_1 = 143.75' \text{ k}$
 $M_2 = 143.75' \text{ k}$

- column b
 $V = 196.67 \text{ k}$
 $N = 47.5 \text{ k}$
 $M_1 = 856.25' \text{ k}$
 $M_2 = 1475.03' \text{ k}$



- column c

$$V = 323.33 \text{ k}$$

$$N = 47.5 \text{ k}$$

$$M_1 = 1331.23 \text{ 'k}$$

$$M_2 = 2425 \text{ 'k}$$

- at support

$$V = 149.2 \text{ k}$$

$$N = 366 \text{ k}$$

$$M_1 = -212.25 \text{ 'k}$$

$$M_2 = 1119 \text{ 'k}$$

15.4.3 Chord Member Sizing

Because the chords are fully laterally braced by the floor diaphragm, the allowable stresses may be taken as $F_a = 0.6F_y = 30 \text{ ksi}$ and $F_b = 0.66F_y = 33 \text{ ksi}$. For members that behave as beams and columns, or in bending and axial loading, as do the chord members, the following formula may be used to determine the required cross-sectional area:

$$A_{req.} = \frac{F + B_x M}{F_a}$$

where B_x is the bending factor $= 2.5/t$ for W-sections, and t is the depth of the section.

- panel A

$$F = 19.2 \text{ k}$$

$$M = 143.75 \text{ 'k}$$

try W12 section

$$A = ((19.2 \text{ k}) + (2.5/12'')(143.75 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 12.62 \text{ in}^2$$

→use W12x45 - $A = 13.2 \text{ in}^2$, $S = 58.1 \text{ in}^3$

For the calculation of the combined effects for the other members, please refer to Appendix 3C.

- panel B

$$F = 419.17 \text{ k}$$

$$M = 856.25 \text{ 'k}$$

try W24 section

$$A = ((419.17 \text{ k}) + (2.5/24'')(856.25 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 49.65 \text{ in}^2$$

→use W24x176 - $A = 51.7 \text{ in}^2$, $S = 450 \text{ in}^3$

- panel C

$$F = 1002.5 \text{ k}$$

$$M = 1331.23 \text{ 'k}$$

try W36 section

$$A = ((1002.5 \text{ k}) + (2.5/36'')(1331.23 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 70.4 \text{ in}^2$$

→use W36x260 - $A = 72.1 \text{ in}^2$, $S = 895 \text{ in}^3$

15.4.4 Column Member Sizing

The same principles as those above apply for the column members as well.

- column a

$$F = 28.75 \text{ k}$$

$$M = 143.75 \text{ 'k}$$

try W12 section

$$A = ((28.75 \text{ k}) + (2.5/12'')(143.75 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 12.94 \text{ in}^2$$

→use W12x45 - $A = 13.2 \text{ in}^2$, $S = 58.1 \text{ in}^3$

- column b

$$F = 47.5 \text{ k}$$

$$M = 1475.03 \text{ 'k}$$

try W27 section

$$A = ((47.5 \text{ k}) + (2.5/27'')(1475.03 \text{ 'k})(12''/'))/(30 \text{ ksi}) = 56.2 \text{ in}^2$$

→use W27x217 - $A = 63.8 \text{ in}^2$, $S = 624 \text{ in}^3$

- column c

$$F = 47.5 \text{ k}$$

$$M = 2425 \text{ 'k}$$

In this case, the moment will clearly control the design, thus, the

required section modulus will be determined with an increase of 10% to account for the axial load.

$$S = (1.1)M/F_b = (1.1)((2425 \text{ 'k})/(33 \text{ ksi}))(12 \text{ '}/\text{'}) = 970 \text{ in}^3$$

→use W33x291 - $A = 85.6 \text{ in}^2$, $S = 1010 \text{ in}^3$

- at support

$$F = 366 \text{ k}$$

$$M = 1119 \text{ 'k}$$

try W30 section

$$A = ((366 \text{ k}) + (2.5/30'')(1119 \text{ 'k})(12 \text{ '}/\text{'})) / (30 \text{ ksi}) = 49.4 \text{ in}^2$$

→use W30x191 - $A = 56.1 \text{ in}^2$, $S = 598 \text{ in}^3$

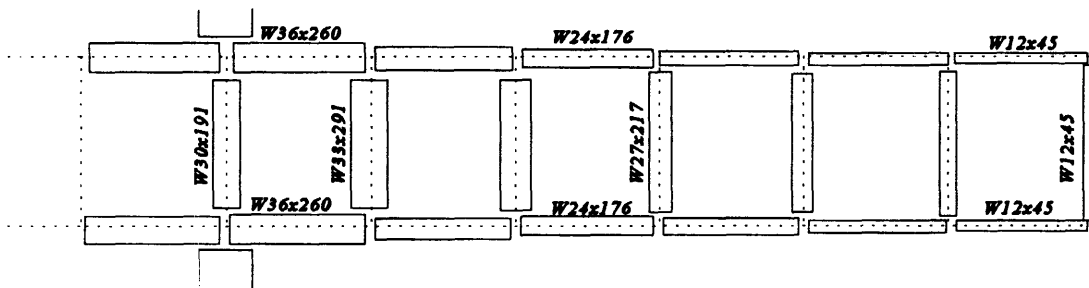


Figure 15.6: General truss member sizes

15.4.5 Truss Deflection Check

The deflection of the truss under gravity loading should be checked to assure that it is within the allowable limits. The maximum deflection allowed under dead and live loading on floor members is given by:

$$\Delta_{all} = L/240 = (60')(12"/')/240 = 3"$$

The deflection of a truss may be estimated in the same way as the lateral deflection of a rigid frame. The equation is as follows:

$$\Delta \sim \frac{2f_a H^2}{3EB}$$

where f_a is the maximum stress in the member at the support, H is the height or length, and B is the distance between the supporting members. Thus:

$$f_a = N/A = (1002.5 \text{ k})/(76.5 \text{ in}^2) = 13.1 \text{ ksi}$$

$$B = 15'$$

$$H = 60'$$

$$\begin{aligned}\Delta &= (2(13.1 \text{ ksi})(60')^2)/(3(29000 \text{ ksi})(15'))(12"/') \\ &= 0.86" < \Delta_{all} \rightarrow \text{OK}\end{aligned}$$

15.5 Determination of Core Thickness

For this scheme, the same procedure may be followed as for scheme 2 in determining the core thickness (refer to section 14.5). The only difference is in the magnitude of the point load transferred by each truss.

$$P = R = 674.69 \text{ k}$$

$$P_D = 75\%P = 506.02 \text{ k}$$

$$P_L = 25\%P = 168.67 \text{ k}$$

$$P_U = 1.4P_D + 1.7P_L = 995.17 \text{ k}$$

- load is bearing on concrete $\rightarrow \phi = 0.7$
 $\rightarrow f'_c = 0.7f'_c = 0.7(8 \text{ ksi}) = 5.6 \text{ ksi}$

$$\begin{aligned} A_{\text{req.}} &= P_U / f'_c = (995.17 \text{ k}) / (5.6 \text{ ksi}) = 177.7 \text{ in}^2 \\ &= 13.33" \times 13.33" \\ &\rightarrow \underline{t_{\text{min}} = 1.25'} \end{aligned}$$

For total dead load, bending, shear, and deflection, the values for t would be the same as those obtained for scheme 2, thus, these calculations will not be repeated here. However, it may be seen that the core thickness for this scheme must be at least 1.25'.

\rightarrow The approximate thickness of the core will be 1.25'

$$A = 220 \text{ ft}^2$$

$$V = (220 \text{ ft}^2)(235') = 51695.5 \text{ ft}^3 = 1915 \text{ CY}$$

15.6 Conclusions

To summarize the results of the investigation of scheme 3, the following structural member sizes were determined:

Chord member A	W12x45
Chord member B	W24x176
Chord member C	W36x260
Column member a	W12x45
Column member b	W27x217
Column member c	W33x291
Column at support	W30x191
Core wall thickness	1.25'

The above values may be used to determine the total dead weight of the building:

Dead load	$= 75\%(28931.5 \text{ k}) = 21698.63 \text{ k}$
Weight per truss	$= (0.045 \text{ k}') (10' / \text{member}) (4 \text{ members/truss})$ $+ (\sim 0.110 \text{ k}') (10' / \text{member}) (8 \text{ members/truss})$ $+ (0.176 \text{ k}') (10' / \text{member}) (4 \text{ members/truss})$ $+ (\sim 0.200 \text{ k}') (10' / \text{member}) (4 \text{ members/truss})$ $+ (0.260 \text{ k}') (10' / \text{member}) (4 \text{ member/truss})$ $+ (0.045 \text{ k}') (15' / \text{member}) (2 \text{ member/truss})$ $+ (\sim 0.160 \text{ k}') (15' / \text{member}) (4 \text{ member/truss})$ $+ (0.217 \text{ k}') (15' / \text{member}) (2 \text{ member/truss})$ $+ (0.260 \text{ k}') (15' / \text{member}) (4 \text{ member/truss})$ $+ (0.291 \text{ k}') (15' / \text{member}) (2 \text{ member/truss})$ $+ (\sim 0.52 \text{ k}') (50')$

	$= 103.83 \text{ k}$
Weight of core	$= (0.150 \text{ kcf})(51695.4 \text{ ft}^3) = 7754.32 \text{ k}$
Total building weight	$= \underline{32360.19 \text{ k}}$

This scheme behaves the same way as scheme 2 in that no torsion is generated strictly from gravity loading. Thus, it also has a much smaller concrete area than scheme 1. However, scheme 3 does require a bit more concrete area than does scheme 2 because of the larger point load on the concrete at each level. Thus, this building, even though it uses much less steel, weighs a slight amount more than scheme 2 due to the extra concrete.

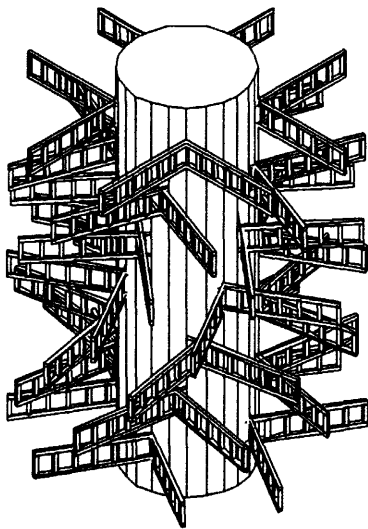


Figure 15.7: General view of vierendeel trusses on alternating floors

CHAPTER 16

Comparison of Three Alternatives and Selection of One

Now that the preliminary investigation of each alternative has been performed, a reasonable evaluation of each and a comparison between the three may be studied. This will allow for the identification of one of the three alternatives for further study. It shall be noted at this time that the following exercise is strictly preliminary by nature of the problem. The monetary values may not reflect actual dollar values but will give a basis for comparison of the three alternatives.

In order to perform even a rough evaluation of cost, dollar values must be assigned to various items. The basis for the comparison here will be the amount of material used (reinforced concrete and structural steel) and the independent construction of the concrete and steel portions of the structures. Dollar values for concrete and steel will be obtained from the *Means Building Construction Cost Data, 1992 edition*. The factors for construction of concrete and steel will be estimated using the same. Note that the construction factors will be rough and will only be utilized to obtain normalized *unit* values for each of the three alternative structural systems. The construction factor will be multiplied by another factor to account for the degree of difficulty of the respective structural systems. The latter factor is strictly a value assigned by the author.

As was mentioned earlier in this paper, A572 Grade 50 steel will be used. Taking into account that larger than standard sections are necessary, the price of steel will be taken as approximately \$700/ton of steel. An estimated base factor of two (2) will be applied to the price of the material to obtain the cost of construction of the steel structural portion, or a total cost of \$2100/ton will be applied to the steel portion of the structure.

For 8000 psi concrete, *Means* gives a rough value of \$200/CY of concrete, including the reinforcement. Assuming slipforming or climbing forms will be used to construct the core, a base factor of three (3) will be applied to the cost of material to obtain the cost of construction of the core, or a total of \$800/CY for the concrete portion of the structure.

Note that the above values do not include the total cost of construction.

Following is a brief discussion of each of the three alternatives and the determination of their relative costs.

16.1 Scheme 1: Suspension System

Structurally, this scheme does not provide an efficient structural solution. The gravity loads alone generate enormous torsional forces on the core and great moments in the floor systems. This results in the requirement of large amounts of structural material; 4452 CY of concrete and 1657.5 ton of steel. The moments induced in the top trusses result in the need for structural steel members that will require special fabrication and thus, will add much cost to the building.

The primary disadvantage of scheme 1 is in the amount of concrete that will be required to resist the torsional component of the gravity loading. This will result in a very massive and heavy structure that, though stable under lateral loading, will exhibit an inefficient use of rentable space and use an enormous amount of concrete material.

As far as construction is concerned, the primary problem would be in constructing the 3-foot thick core. Slipforming will be a possible alternative for the construction of the core though slipforming walls 3 feet thick is not the common practice. Thus, the construction of the core may result in high costs. The core will most likely require much more reinforcement than those of scheme 2 and 3 because of the large torsional stresses. However, the core wall will require only minimum penetrations for the support of the floor beams since they need not penetrate through the core. A value of 1.25 will be assigned as the degree of difficulty factor for construction of the core for scheme 1 (As will be seen shortly, this value represents a degree of difficulty of 25% more than the simplest construction of scheme 3).

Erecting the steel structure will most likely be a straight forward process beginning with the placement of the trusses and then suspending the floors one at a time from the top down. The trusses will be the most difficult to erect into position. Most likely, they will require shop fabrication of small panels to be shipped to the site, at which time they may be connected into larger panels for lifting, and then finally lifted into place and connected to form the continuous truss. The beams may be easily transported to the site in their final form and be ready for lifting into place. However, alignment of the tension members will be very difficult; achievement of the desired form will be very sensitive to accurate placement of the beams and tension members. A value of 1.2 will be assigned as the degree of difficulty factor

for construction of steel for scheme 1.

<i>Scheme 1</i>	<i>units</i>	<i>mat. cost factor</i>	<i>material cost,\$</i>	<i>const. cost factor</i>	<i>construction cost,\$</i>	<i>total cost,\$</i>
<i>concrete</i>	<i>4452CY</i>	<i>200\$/CY</i>	<i>890,400</i>	<i>3x1.25 =3.75</i>	<i>3,339,000</i>	<i>4,229,400</i>
<i>steel</i>	<i>1657.5ton</i>	<i>700\$/ton</i>	<i>1,160,250</i>	<i>2x1.2 =2.4</i>	<i>2,784,600</i>	<i>3,944,850</i>
<i>totals</i>			<i>\$2,050,650</i>		<i>\$ 6,123,600</i>	<i>\$8,174,250</i>

Note that this is not the total cost of the building but rather a rough cost to use as a relative measure against alternatives 2 and 3.

16.2 Scheme 2: Vierendeel Truss System

Structurally, this scheme is much more efficient than scheme 1 in that no additional forces other than vertical are generated from the gravity loading. However, some inefficiency does exist in that, because the trusses are repeated at every floor, the floor loads, especially in the lobby areas, are carried by two systems when only one system is necessary; a great redundancy exists in the system. Furthermore, the overlapping of adjacent trusses will create difficult connections and thus, difficult fabrication conditions.

The amounts of material used are quite reasonable; total concrete weight is 1540 CY and the total steel weight is 1704.5 ton. The minimum amount of concrete material is required for vertical and lateral stability of the structure. For construction, slipforming or climbing forms will be very suitable for the construction of the core with allowances made during the process for the erection of the trusses upon completion of the core. However, the core will require a large number of penetrations to accommodate the penetration of the trusses at *every* floor level. This will cause great difficulty in the forming process. The degree of difficulty of concrete construction will be 1.15 for scheme 2.

The fabrication of the trusses may be accomplished in a similar fashion of those in scheme 1 above and then raised one at a time in sections, with the middle section being lowered through the core and the two outer sections being raised around the core. Again, much care must be taken in the alignment of the trusses so that the spiral form will be achieved. A value of 1.1 will be assigned to the degree of difficulty factor for steel construction.

<i>Scheme 2</i>	<i>units</i>	<i>mat. cost factor</i>	<i>material cost,\$</i>	<i>const. cost factor</i>	<i>construction cost,\$</i>	<i>total cost,\$</i>
<i>concrete</i>	<i>1540CY</i>	<i>200\$/CY</i>	<i>308,000</i>	<i>3x1.15 =3.45</i>	<i>1,062,600</i>	<i>1,370,600</i>
<i>steel</i>	<i>1704.5ton</i>	<i>700\$/ton</i>	<i>1,193,150</i>	<i>2x1.1 =2.2</i>	<i>2,624,930</i>	<i>3,818,080</i>
<i>totals</i>			<i>\$1,501,150</i>		<i>\$ 3,687,530</i>	<i>\$5,188,680</i>

Note that this scheme has a cost only a bit greater than one-half that of scheme 1.

16.3 Scheme 3: Alternating Vierendeel Truss System

Structurally, this scheme is also much more efficient than scheme 1 in that no additional forces other than vertical are generated from the gravity loading. However, it is also more efficient than scheme 2 in that the trusses are repeated only every other floor so that each truss is used to its fullest potential in all sections. Thus, the difficulties of fabricating the joint details that exist in scheme 2 are not present in scheme 3.

Scheme 3 uses a bit more concrete than scheme 2, however, the steel amount for scheme 3 is less; total concrete weight is 1915 CY and the total steel weight is 1345.5 ton. For construction, slipforming or climbing forms will be very suitable for the construction of the core with allowances made during the process for the erection of the trusses upon completion of the core. Penetrations for the trusses are only necessary on every other floor,

making the construction of the core less complex than that of scheme 2. A value of 1.0 will be assigned to the degree of difficulty factor for construction of concrete.

The trusses may be raised one at a time in sections, with the middle section being lowered through the core and the two outer sections being raised around the core. This process is identical to that of scheme 2, however, scheme 3 requires only half as many trusses to be lifted. Again, much care must be taken in the alignment of the trusses so that the spiral form will be achieved, however, the erection process is less complex than those of schemes 1 and 2. The degree of difficulty factor for steel construction will be 1.0.

<i>Scheme 3</i>	<i>units</i>	<i>mat. cost factor</i>	<i>material cost, \$</i>	<i>const. cost factor</i>	<i>construction cost, \$</i>	<i>total cost, \$</i>
<i>concrete</i>	<i>1915CY</i>	<i>200\$/CY</i>	<i>383,000</i>	<i>3x1.0 =3.0</i>	<i>1,149,000</i>	<i>1,532,000</i>
<i>steel</i>	<i>1345.5ton</i>	<i>700\$/ton</i>	<i>948,150</i>	<i>2x1.0 =2.0</i>	<i>1,896,300</i>	<i>2,844,450</i>
<i>totals</i>			<i>\$1,331,150</i>		<i>\$ 3,045,300</i>	<i>\$4,376,450</i>

16.4 Comparison of Three Schemes and Selection of One

Though it was mentioned previously that a suspension system usually provides a more efficient use of steel, this was not the outcome for scheme 1. In fact, scheme 1 actually utilizes almost as much steel material as schemes 2 and more than scheme 3. The design of the suspension system will be quite complex and time-consuming, and the construction will also require much time and specialized skill, techniques, and equipment. In all, scheme 1 proved to be the most inefficient and expensive system. Scheme 2 has much fewer difficulties involved in the design and construction of the structure as compared to scheme 1. However, as compared to scheme 3, scheme 2 will require a more intensive construction process

because of the number of trusses to be fabricated and erected, nearly double that of scheme 3. Furthermore, the joining of adjacent trusses will cause some difficulty in the construction, resulting in the total cost determined above. Scheme 3, based on the above analysis, is the most desirable one of the three. It contains the fewest elements to be fabricated and erected and has the simplest system for analysis and design, as well as being the least expensive of the three alternatives.

For comparative purposes only, the relationships between the cost of the three alternatives may be helpful. This will be accomplished simply by dividing the above total costs by the lowest cost value of \$4,376,450. Thus, it will be possible to determine the percent difference between the systems.

Scheme 1	Scheme 2	Scheme 3
1.87	1.20	1.0

Thus, the most optimum solution is scheme 3, the *vierendeel truss system on alternating floors*. This system provides the most efficient use of materials with the least redundancy in structural elements and the lowest cost.

CHAPTER 17

Conclusion: Structural System for the Spiral Building

By now, the reader should have a general understanding of the processes involved in the preliminary design of a tall building. It has been discussed that there is much more to the design of a tall building structure than just "number-crunching"; there is a great deal of consideration and planning that must be done before the design phase may begin. Such items that require consideration prior to the selection of a structural system include building use and occupancy, geographical location, special loading conditions, availability of specific materials and qualified construction equipment and personnel locally or otherwise, and accessibility to the site for delivery of special materials and equipment. One of the most important issues to be considered is construction and constructibility; the availability of construction techniques, equipment, and labor in the area and the availability of time for construction, all of which have an impact on any structural design. This is an issue that designers often fail to consider, though the consequences of construction on a design may be great.

The analyses performed in this paper for the *Spiral Building* do not take all of the above-mentioned considerations into account. This is because the purpose here was to investigate a building form to be applied to a general set of constraints rather than attempting to find a solution *given* a specific set of constraints, as would be the case in a real-life situation. However, the results obtained in this paper may be modified as necessary to accommodate any specific conditions or requirements that may be placed on the *Spiral Building*; the results here are general and represent only an idea for the solution of the *Spiral Building*.

The general structural system selected for the *Spiral Building* is a core structure with the floors hung off the core. This type of system is quite common, as was seen in the chapters on *Precedent Buildings*. Furthermore, it was seen that there do exist buildings similar in form to the *Spiral Building*.

The investigation of the *Spiral Building* consisted of the analyses of three structural schemes, each of which were found to be viable solutions and could be fully designed to effectively produce the desired result. This reinforces the fact that a single building form may be accommodated by any of a variety of structural systems, each with its own advantages and disadvantages. As is done in practice, the three schemes were compared so that the optimum solution may be obtained. Basis for comparison included quantity of material necessary, ease

of design and analysis, and ease of construction. These items were all weighed and the combination of them yielded a *relative* cost value for each alternative.

In performing the analysis, it was found that scheme 1, as a suspension system, is not efficient. A suspension system typically allows for a substantially smaller amount of steel than does a typical steel structure, thus reducing the cost of necessary material as well as the weight of the structure. However, scheme 1 actually requires nearly as much steel as scheme 2, which requires the largest amount of steel from the three alternatives. Furthermore, scheme 1 requires an large amount of concrete. Though a deflection analysis was not performed for scheme 1, this issue would be of major concern in a more in-depth design of the structure, as would be flexibility problems, thus making the design process complex. The construction process for such a system would also be complex because of the thickness of the core wall and the sensitivity of the suspension system to accurate erection.

In relating only the material usage, scheme 2 is more efficient than scheme 1 because, though scheme 2 requires only slightly more steel, it requires much less concrete than scheme 1. Furthermore, scheme 2 will have much less of a flexibility problem; in fact, the structure will be quite stiff because of the interaction of adjacent floors due to their being connected. In other words, the entire height of the building floors will act as a unit and will thus, create a stiff resistance to loads and vibration. As far as construction of the structure, the process would be relatively straight-forward, except for some possible difficulties in the connection of the adjacent trusses. The design and analysis are quite direct and standard, and so, scheme 2 is by far a more effective structural system than that of scheme 1 in its use of material and in its behavior.

Scheme 3 was found to be the most efficient system of the three alternatives considered. The structure is the least complicated to analyze and design and the construction of the structure is the most direct.

The comparison of the three alternatives in chapter 16 yielded scheme 3, the *Story-high Vierendeel Trusses on Alternating Floors*, to be the optimum one. A preliminary design of the structural system has been obtained in chapter 15. However, the final design for the *Spiral Building* is far from complete at this stage; a complete design will require a great deal more analysis. For example, torsional effects due to wind acting on the building form were

not considered and vertical deflection of the floors were not looked at in any detail. The investigation of the structure for the *Spiral Building* will end here, though it has been proven that a final and effective design may be achieved.

In the following sections, some functional requirements of the building will be briefly discussed.

PART 5

Functional Requirements

CHAPTER 18

Functional Requirements and Space Planning

This chapter will briefly discuss the functional requirements of a tall residential building as based on the Uniform Building Code.

18.1 Exit Requirements

The building codes are very specific about the requirements for egress from a building or a portion of a building. The requirements include size of exits (doors and stairs), locations of exits, and number of exits. Following, is a brief investigation of the exit requirements for the *Spiral Building*.

18.1.1 Occupancy Loads

Occupancy loads are necessary for the determination of the sizes and number of exits required. From Chapter 33, table 33-A of the UBC, the occupancy load factors are:

Living units (#15)	1/200
Mechanical room (#21)	1/300
Storage room (#26)	1/300
Lounge (#4)	1/15
Recreation room (#4)	1/15
Exercise room (#12)	1/50
Ground floor lobby (#30)	1/100
Garage (#13)	1/200

Multiplying the occupancy load factors by the areas of the occupancies yields the occupancy loads:

Living units	$1/200(2665 \text{ ft}^2) = 13$
--------------	---------------------------------

Mechanical room	$1/300(3848.5 \text{ ft}^2) = 13$
Storage room	$1/300(2665 \text{ ft}^2) = 9$
Lounge	$1/15(2665 \text{ ft}^2) = 178$
Recreation room	$1/15(2665 \text{ ft}^2) = 178$
Exercise room	$1/50(2665 \text{ ft}^2) = 54$
Ground floor lobby	$1/100(3848.5 \text{ ft}^2) = 39$
Garage	$1/200(30000 \text{ ft}^2) = 150$

Thus, the total occupancy loads per floor are:

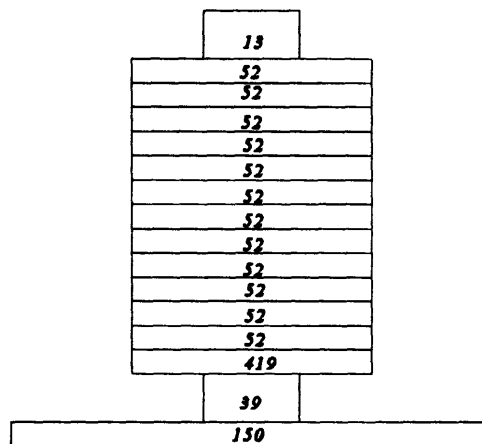


Figure 18.1: Occupancy loads

18.1.2 Required Exits

The size of the exits depends on the occupancy load that the particular exit serves. The formula for determining the occupancy load served by an exit is as follows:

The total occupancy load for an exit on a given floor is the occupancy load of the floor plus 50% of the occupancy load of floor directly above plus 25% of the

occupancy load of the next floor above plus 50% of the occupancy load of the floor directly beneath if it exits into the floor under consideration.

Thus, the loads are:

Floor 14	13
Floor 13	$52 + 50\%(13) = 59$
Floor 12	$52 + 50\%(52) + 25\%(13) = 81$
Floor 11-2	$52 + 50\%(52) + 25\%(52) = 91$
Floor 1	$419 + 50\%(52) + 25\%(52) = 458$
Ground floor	$39 + 50\%(419) + 25\%(52) + 50\%(150) = 336$

The required stair width is given by $w = 0.3 \times \text{occupancy load}$ or 44", whichever is greater.

Floor 11-2	$w = 0.3(91) = 27.3" \rightarrow 44" \rightarrow 4'$
Floor 1	$w = 0.3(458) = 137.4" \rightarrow 11.5'$

The required door width is given by $w = 0.2 \times \text{occupancy load}$ or 36", whichever is greater.

Ground floor	$w = 0.2(336) = 67.2" \rightarrow 5.6'$
--------------	---

A minimum of two exits are required for occupancy loads greater than 10.

18.2 Possible Lobby Layout

From the above calculations and with reference to the UBC, 4' minimum stair width for floors 2-13 and 11.5' minimum stair width for floor 1 are required. Furthermore, a minimum of two exit stairs are required at all floors. This may be accomplished by placing a

pair of scissor stairs in the service core throughout the height of the building and placing an additional exit stair from the first floor to the ground level. This is illustrated in figure 18.1. All stairs are 4 feet wide with the minimum required rise-to-run of 7" to 11".

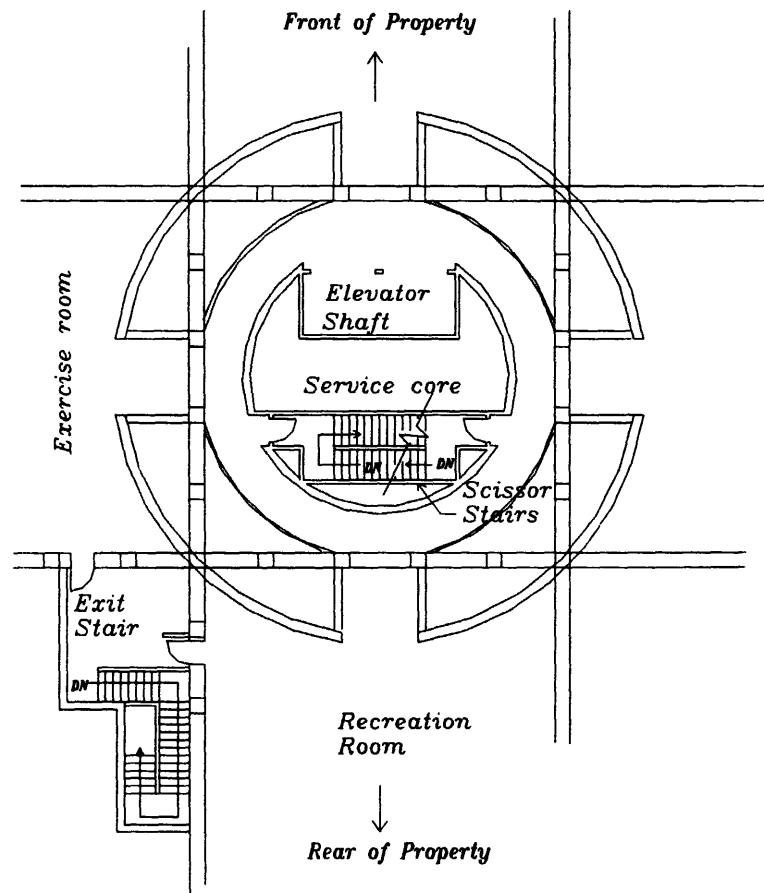


Figure 18.1: Lobby layout at first floor

The service core will also house all the vertical mechanical systems such as plumbing, and other services such as trash chutes, etc. These systems will be brought into and out of the units through the penetration in the core that allows access into the units.

18.3 Some Possible Unit Floor Plans

The following figures illustrate some possible floor plan alternatives for the units. Note in figure 18.3 the sense of privacy that is achieved by including windows only on two faces. Furthermore, it should be noted that the possibilities for floor layouts are many because of the freedom of space achieved from the structure being confined to only the perimeter limits of the spaces.

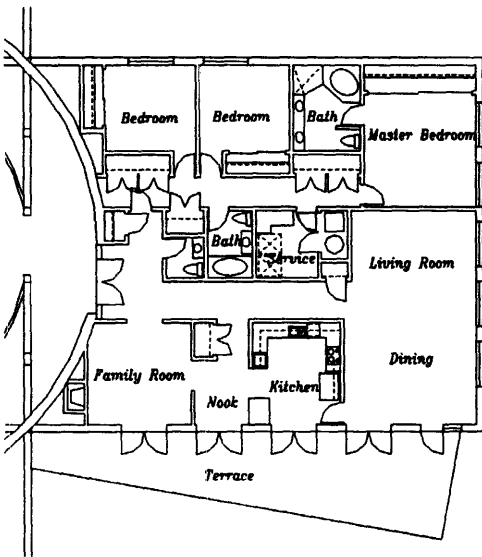


Figure 18.2: Floor plan alternative

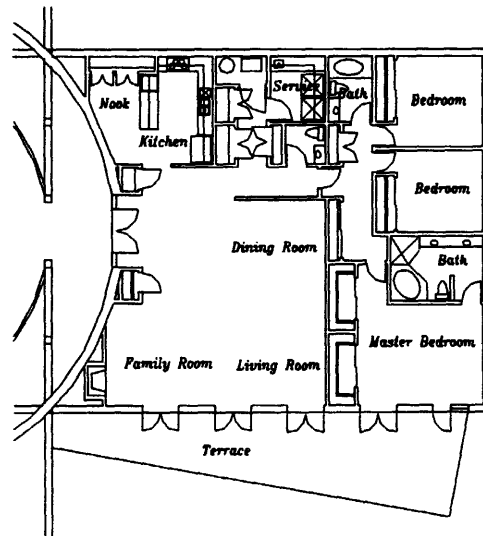


Figure 18.3: Floor plan alternative

APPENDICES

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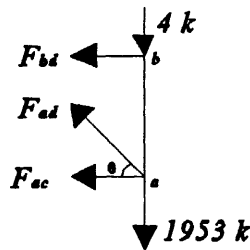
Appendix 1

Appendix 1A

Scheme 1 - Calculation of axial forces in truss members

The axial forces in the truss members will be investigated using the method of sections. Only three sections shall be analyzed.

- section A-A



$$\Sigma F_y = F_{ad} \sin \theta - 1953 \text{ k} - 4 \text{ k} = 0$$

$$\rightarrow F_{ad} = 2253 \text{ k}$$

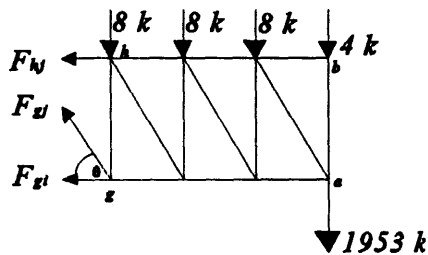
$$\Sigma M_a = F_{bd}(15') = 0$$

$$\rightarrow F_{bd} = 0 \text{ k}$$

$$\Sigma F_x = F_{bd} + F_{ad} \cos \theta + F_{ac} = 0$$

$$\rightarrow F_{ac} = -1057.5 \text{ k}$$

- section B-B



$$\Sigma F_y = F_{gj} \sin \theta - 1953 \text{ k} - 28 \text{ k} = 0$$

$$\rightarrow F_{gj} = 2280 \text{ k}$$

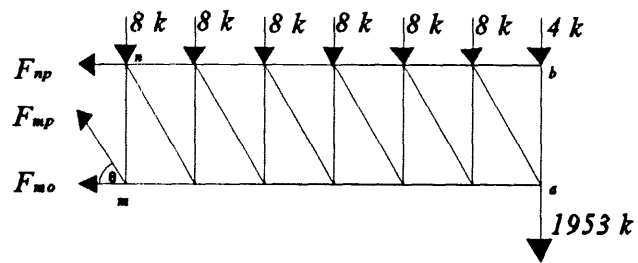
$$\Sigma M_g = F_{hj}(15') - (8 \text{ k})(8' + 16') - (4 \text{ k})(24') - (1953 \text{ k})(24') = 0$$

$$\rightarrow F_{hj} = 3195.2 \text{ k}$$

$$\Sigma F_x = F_{hj} + F_{gi} \cos \theta + F_{gj} = 0$$

$$\rightarrow F_{gi} = -4265.6 \text{ k}$$

- section C-C



$$\Sigma F_y = F_{mp} \sin \theta - 1953 \text{ k} - 52 \text{ k} = 0$$

$$\rightarrow F_{mp} = 2307 \text{ k}$$

$$\Sigma M_m = F_{np}(15') - (8 \text{ k})(8' + 16' + 24' + 32' + 40') - (4 \text{ k})(48') - (1953 \text{ k})(48') = 0$$

$$\rightarrow F_{np} = 6428.8 \text{ k}$$

$$\Sigma F_x = F_{np} + F_{mp} \cos \theta + F_{mo} = 0$$

$$\rightarrow F_{mo} = -7512 \text{ k}$$

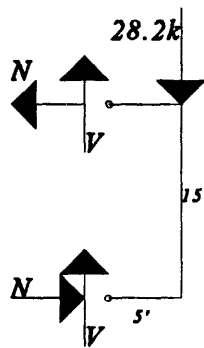
Appendix 2

Appendix 2A

Scheme 2 - Calculation of truss member forces

The axial and shear forces and the moments of the truss members will be investigated by the portal method. It will be assumed that full frame action exists so that hinges, or points of zero-moment, will occur at mid-height and mid-span of the members. The free bodies are taken by cutting the panels at the hinges so it will be ensured that only shear and axial forces will occur. This method is commonly used for the preliminary design of frames and trusses.^{1,2}

- panel A

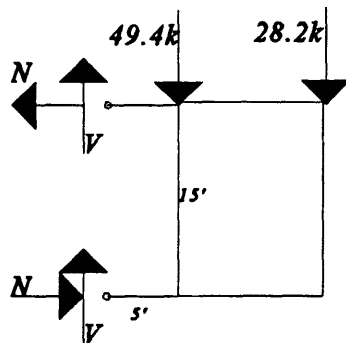


$$\Sigma F_y = 2V - 28.2k = 0$$

$$\rightarrow V = 14.1k$$

$$\Sigma M = N(15') - (28.2k)(5')$$

$$\rightarrow N = 9.4k$$



$$\Sigma F_y = 2V - 28.2k - 49.4k = 0$$

$$\rightarrow V = 38.8k$$

$$\Sigma M = N(15') - (49.4k)(5') - (28.2k)(15') = 0$$

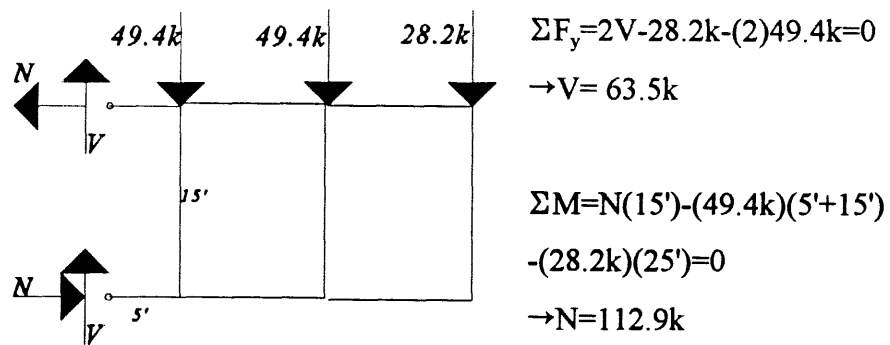
$$\rightarrow N = 44.7k$$

1

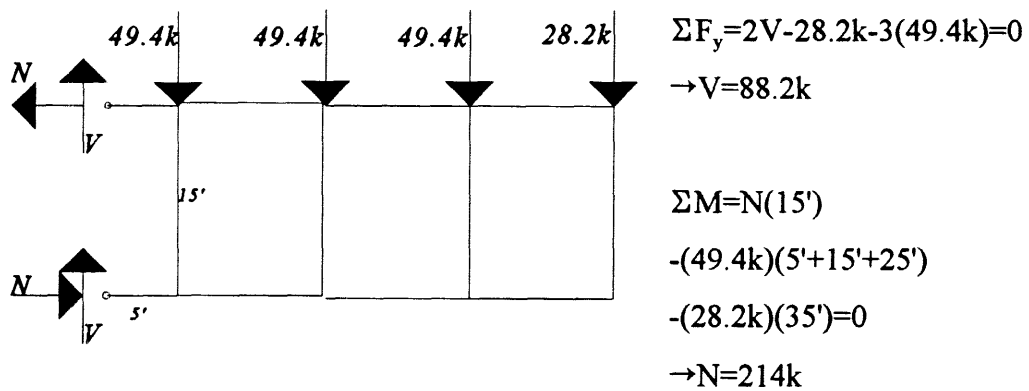
Schueller, W., High-rise Building Structures, Second edition, John Wiley and Sons, Inc., 1977.

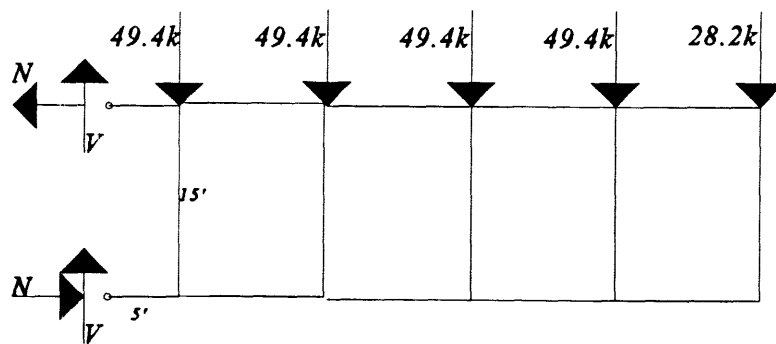
2

Hsieh, Yuan-Yu, Elementary Theory of Structures, Third edition, Prentice Hall, 1988.



• panel B





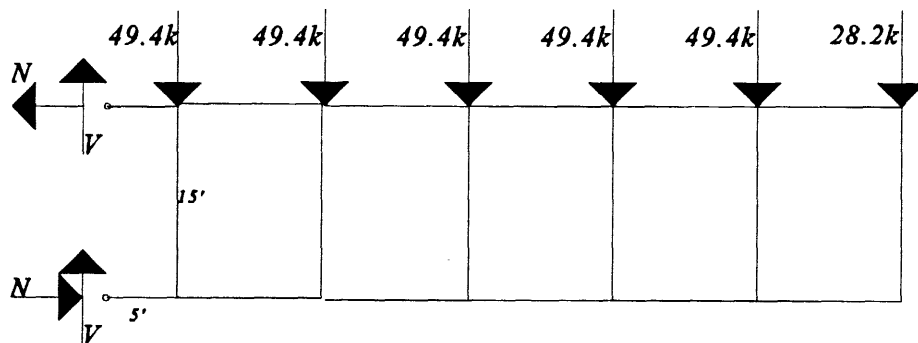
$$\Sigma F_y = 2V - 28.2k - 4(49.4k) = 0$$

$$\rightarrow V = 112.9k$$

$$\Sigma M = N(15') - (49.4k)(5' + 15' + 25' + 35') - (28.2k)(45') = 0$$

$$\rightarrow N = 348.1k$$

- panel C



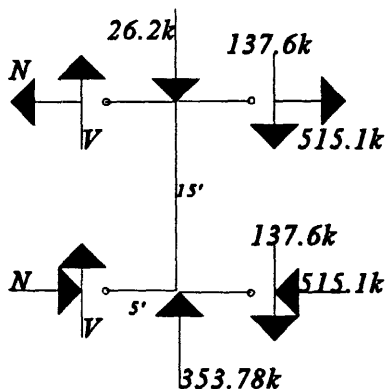
$$\Sigma F_y = 2V - 28.2k - 5(49.4k) = 0$$

$$\rightarrow V = 137.6k$$

$$\Sigma M = N(15') - (49.4k)(5' + 15' + 25' + 35' + 45') - (28.2k)(55') = 0$$

$$\rightarrow N = 515.1k$$

- at support



$$\Sigma F_y = 2V - 26.2k - 2(137.6k) + 353.78k = 0$$

$$\rightarrow V = -26.2k$$

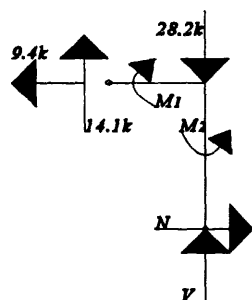
$$\Sigma M = N(15') - (26.2k - 353.78k)(5') -$$

$$2(137.6k)(10') - (515.1k)(15') = 0$$

$$\rightarrow N = 589.3k$$

The shears and axial forces in the columns and the moments in the chords and the columns will be found by cutting the free bodies through the column members.

- column a



$$\Sigma F_x = V - 9.4k = 0$$

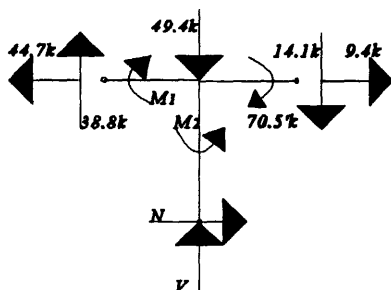
$$\rightarrow V = 9.4k$$

$$\Sigma F_y = N + 14.2k - 28.2k = 0$$

$$\rightarrow N = 14.1k$$

$$M_1 = 14.1k(5') = 70.5'k$$

$$M_2 = 9.4k(7.5') = 70.5'k$$



$$\Sigma F_x = V - 9.4k + 44.7 = 0$$

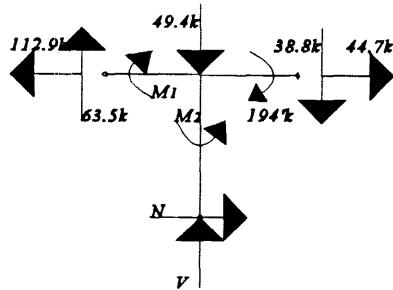
$$\rightarrow V = 35.5k$$

$$\Sigma F_y = N - 14.2k + 38.8k - 49.4k = 0$$

$$\rightarrow N = 24.7k$$

$$M_1 = 38.8k(5') = 194'k$$

$$M_2 = 35.3k(7.5') = 264.75'k$$



$$\Sigma F_x = V - 112.9k + 44.7k = 0$$

$$\rightarrow V = 68.2k$$

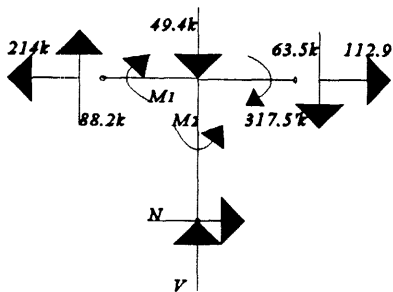
$$\Sigma F_y = N - 49.4k - 38.8k + 63.5k = 0$$

$$\rightarrow N = 24.7k$$

$$M_1 = 63.5k(5') = 317.5'k$$

$$M_2 = 68.2k(7.5') = 511.5'k$$

• column b



$$\Sigma F_x = V - 214k + 112.9k = 0$$

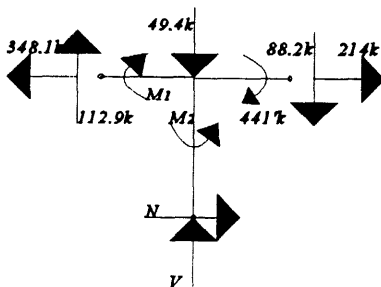
$$\rightarrow V = 101.1k$$

$$\Sigma F_y = N - 49.4k - 63.5k + 88.2k = 0$$

$$\rightarrow N = 24.7k$$

$$M_1 = 88.2k(5') = 441'k$$

$$M_2 = 101.1k(7.5') = 758.5'k$$



$$\Sigma F_x = V - 348k + 214k = 0$$

$$\rightarrow V = 134.1k$$

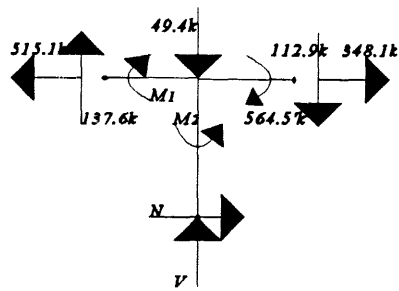
$$\Sigma F_y = N - 49.4k - 88.2k + 112.9k = 0$$

$$\rightarrow N = 24.7k$$

$$M_1 = 112.9k(5') = 564.5'k$$

$$M_2 = 134.1k(7.5') = 1005.75'k$$

- column c



$$\Sigma F_x = V - 515.1k + 348.1k = 0$$

$$\rightarrow V = 167k$$

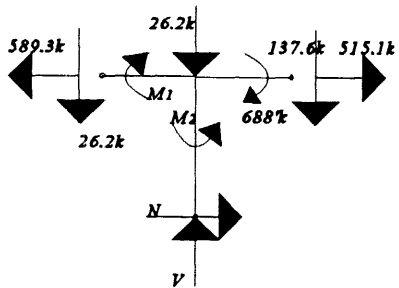
$$\Sigma F_y = N - 49.4k - 112.7k + 137.6k = 0$$

$$\rightarrow N = 24.7k$$

$$M_1 = 137.6k(5') = 688'k$$

$$M_2 = 167k(7.5') = 1252.5'k$$

- at support



$$\Sigma F_x = V - 589.3k + 515.1k = 0$$

$$\rightarrow V = 74.2k$$

$$\Sigma F_y = N - 26.2k - 26.2k - 137.6k = 0$$

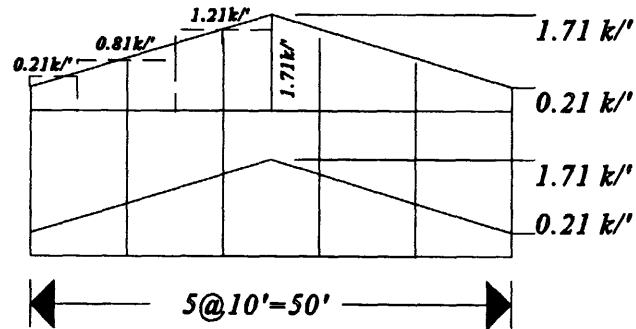
$$\rightarrow N = 190k$$

$$M_1 = -26.2k(5') = -131'k$$

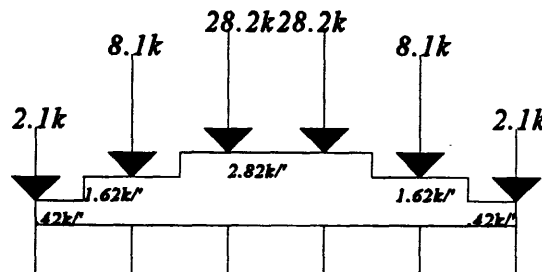
$$M_2 = 74.2k(7.5') = 556.5'k$$

Appendix 2B

Scheme 2 - Determination of point loads in lobbies



From geometry, the stepped values of the triangular loading may be obtained, as is shown above. Doubling these values to account for the two-floor loading and multiplying by the tributary widths of the truss joints, the following point loads are obtained:



Appendix 2C

Scheme 2 - Combined effect of compression and bending on truss members

$$f_a/F_a + f_b/F_b \leq 1$$

- panel B

$$f_a = 214\text{k}/42.7\text{in}^2 = 5.01 \text{ ksi}$$

$$f_b = 441\text{'k}(12\text{'})/232\text{in}^3 = 22.8 \text{ ksi}$$

$$(5.01 \text{ ksi})/(30 \text{ ksi}) + (22.8 \text{ ksi})/(33 \text{ ksi}) = 0.86 < 1$$

- panel C

$$f_a = 515.1\text{k}/47.7\text{in}^2 = 10.8 \text{ ksi}$$

$$f_b = 688\text{'k}(12\text{'})/414\text{in}^3 = 19.9 \text{ ksi}$$

$$(10.8 \text{ ksi})/(30 \text{ ksi}) + (19.9 \text{ ksi})/(33 \text{ ksi}) = 0.96 < 1$$

- column a

$$f_a = 14.1\text{k}/7.61\text{in}^2 = 1.85 \text{ ksi}$$

$$f_b = 70.5\text{'k}(12\text{'})/27.9\text{in}^3 = 30.32 \text{ ksi}$$

$$(1.85 \text{ ksi})/(30 \text{ ksi}) + (30.32 \text{ ksi})/(33 \text{ ksi}) = 0.98 < 1$$

- column b

$$f_a = 24.7\text{k}/46.3\text{in}^2 = .53 \text{ ksi}$$

$$f_b = 758.5\text{'k}(12\text{'})/310\text{in}^3 = 29.36 \text{ ksi}$$

$$(0.53 \text{ ksi})/(30 \text{ ksi}) + (29.36 \text{ ksi})/(33 \text{ ksi}) = 0.91 < 1$$

- column c

$$f_a = 24.7\text{k}/47\text{in}^2 = 0.52 \text{ ksi}$$

$$f_b = 1252.5'\text{k}(12"/')/542\text{in}^3 = 27.7 \text{ ksi}$$

$$(0.52 \text{ ksi})/(30 \text{ ksi}) + (27.7 \text{ ksi})/(33 \text{ ksi}) = 0.86 < 1$$

- at support

$$f_a = 190\text{k}/30.6\text{in}^2 = 6.2 \text{ ksi}$$

$$f_b = 556.5'\text{k}(12"/')/258\text{in}^3 = 25.9 \text{ ksi}$$

$$(6.2 \text{ ksi})/(30 \text{ ksi}) + (25.9 \text{ ksi})/(33 \text{ ksi}) = 0.99 < 1$$

Appendix 3

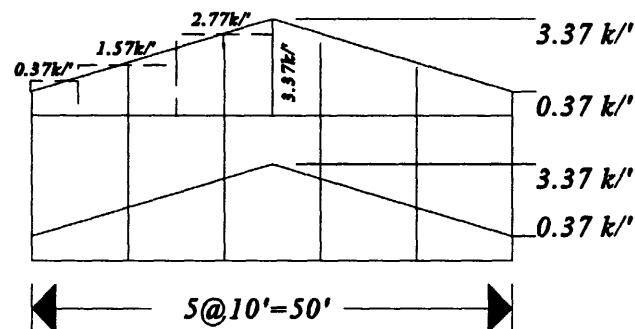
Appendix 3A

Scheme 3 - Determination of truss member forces

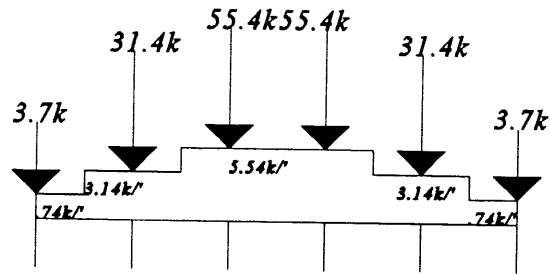
The calculations for the forces in the truss members for scheme 3 are identical to those of scheme 2 except that the values of the loads are greater and thus, the magnitudes of the forces are larger. The exact calculations will not be performed here. Refer to the calculation above in Appendix 2A for the procedure of determining the forces.

Appendix 3B

Scheme 3 - Determination of point loads on lobbies



From geometry, the stepped values of the triangular loading may be obtained, as is shown above. Doubling these values to account for the two chords of the truss and multiplying by the tributary widths of the truss joints, the following point loads are obtained:



Appendix 3C

Scheme 3 - Combined effect of compression and bending on truss members

$$f_a/F_a + f_b/F_b \leq 1$$

- panel A

$$f_a = 19.2k/13.2in^2 = 1.45 \text{ ksi}$$

$$f_b = 143.75k(12"/ft)/581in^3 = 29.69 \text{ ksi}$$

$$(1.45 \text{ ksi})/(30 \text{ ksi}) + (29.69 \text{ ksi})/(33 \text{ ksi}) = 0.95 < 1$$

- panel B

$$f_a = 419.17k/51.7in^2 = 8.1 \text{ ksi}$$

$$f_b = 856.25k(12"/ft)/450in^3 = 22.8 \text{ ksi}$$

$$(8.1 \text{ ksi})/(30 \text{ ksi}) + (22.8 \text{ ksi})/(33 \text{ ksi}) = 0.96 < 1$$

- panel C

$$f_a = 1002.5k/72.1in^2 = 13.9 \text{ ksi}$$

$$f_b = 1331.23 \text{ k}(12 \text{ in})^3 / 895 \text{ in}^3 = 17.85 \text{ ksi}$$

$$(13.9 \text{ ksi}) / (30 \text{ ksi}) + (17.85 \text{ ksi}) / (33 \text{ ksi}) = 1 < 1$$

- column a

$$f_a = 28.75 \text{ k} / 13.2 \text{ in}^2 = 2.18 \text{ ksi}$$

$$f_b = 143.75 \text{ k}(12 \text{ in})^3 / 581 \text{ in}^3 = 29.7 \text{ ksi}$$

$$(2.18 \text{ ksi}) / (30 \text{ ksi}) + (29.7 \text{ ksi}) / (33 \text{ ksi}) = 0.97 < 1$$

- column b

$$f_a = 47.5 \text{ k} / 63.8 \text{ in}^2 = 0.74 \text{ ksi}$$

$$f_b = 1475.03 \text{ k}(12 \text{ in})^3 / 624 \text{ in}^3 = 28.4 \text{ ksi}$$

$$(0.74 \text{ ksi}) / (30 \text{ ksi}) + (28.4 \text{ ksi}) / (33 \text{ ksi}) = 0.88 < 1$$

- column c

$$f_a = 47.5 \text{ k} / 85.6 \text{ in}^2 = 0.55 \text{ ksi}$$

$$f_b = 2425 \text{ k}(12 \text{ in})^3 / 1010 \text{ in}^3 = 28.8 \text{ ksi}$$

$$(0.55 \text{ ksi}) / (30 \text{ ksi}) + (28.8 \text{ ksi}) / (33 \text{ ksi}) = 0.89 < 1$$

- at support

$$f_a = 366 \text{ k} / 56.1 \text{ in}^2 = 6.5 \text{ ksi}$$

$$f_b = 1119 \text{ k}(12 \text{ in})^3 / 598 \text{ in}^3 = 22.45 \text{ ksi}$$

$$(6.5 \text{ ksi}) / (30 \text{ ksi}) + (22.45 \text{ ksi}) / (33 \text{ ksi}) = 0.90 < 1$$

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